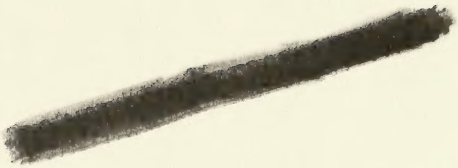

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(INSTITUTED 1852.)

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ERRATA.

APRIL, 1886.—Discussion by E. L. CORTHELL on the South Pass Jetties.

Page 270, 7th line.—Insert the word "not" after the words "an engineer who has." The sentence will then read: "An engineer who has not pursued his way through these schools and obtained his rank, has to remain ten years, often ten or fifteen a 'Conductor,' and then to pass an extraordinary examination in order to be titled 'Ordinary Engineer.'"

Same page, 28th line.—After the words "from among the," insert the word "Conductors" in place of the words "Civil Engineers."

Same page.—After the 29th line insert the following paragraph (the final paragraph remaining as printed):

"It must be confessed that the high rank of the Corps des Ponts et Chaussées must be attributed to the system of admission through the best schools and admitting only the best pupils. This system is a democratic system, its positions being given not as a privilege to rich or noble people, but to the most able young men. Nevertheless, I believe that the doors are not enough open to such engineers as succeed in educating themselves in other ways with the intention of obtaining governmental positions. In France the tendency to be an employé of the government is a general one. I believe it is not so in the United States, and it proves that your people have more feeling for independence, and it is better for you."

JUNE, 1886.—Discussion by GEORGE H. PEGRAM. Page 477. In Table, for the word *Mogul*, read *Class M*. Page 479. Table for Case 3. Third line, fourth column, for 205 414, read 295 414.

JULY, 1886.—Johnson on the Strength of Columns. Page 522, in equations (6), (7) and (8), and page 523, in equations (15), (16) and (17), instead of the letter *l*, there should be the figure 1.

JULY, 1886.—Discussion on The Strength of Columns, by JAMES CHRISTIE, M. Am. Soc. C. E. Table on Page 533. The first line referring to hard steel reads:

Hard steel.....	$1\frac{1}{2}\times1\frac{1}{2}\times\frac{17}{64}$	42	1.25	48 300	38 610	93

This should read as follows:

Hard steel.....	$1\frac{1}{4}\times1\frac{1}{4}\times\frac{17}{64}$	42	1.25	48 300	38 610	114

Mr. Christie states that the error was made originally in making the report of the tests and not discovered till in print.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XV.—January, 1886.)

ENGLISH AND AMERICAN RAILROADS COMPARED.

By EDWARD BATES DORSEY, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE 24TH, 1885.

WITH ADDITIONS READ OCTOBER 7TH, 1885.

Perhaps the first thing that strikes an American engineer in railroad traveling in England * is the inconvenience of the English * passenger car. It should, however, be considered that this style of car suits the exclusive and retiring character of the English; and also that very long journeys, such as we are accustomed to make, are impossible in such a small island; moreover, the English do not travel nearly as much as we do, consequently the confinement in a small, locked-up

* EXPLANATORY NOTE.—In this paper, unless otherwise stated,

The words England or English will represent the United Kingdom, *i. e.*, England, Scotland, Wales and Ireland.

The pound sterling will be considered equal to five dollars American money.

The American ton contains 2 000 pounds.

The English ton contains 2 240 pounds.

The data used in this paper relating to the English railroads, were taken from the reports of the Board of Trade of the United Kingdom, or from the reports of the different railroad companies.

The data relating to the American railroads were taken from the reports of the Railroad Commissioners of the different States, Poor's Manual, and the reports of the different railroad companies.

For some unexplained reason, these different authorities do not always agree, consequently there may be apparent errors in this paper, when comparison is made with authority different from that from which it was originally taken.

compartment is not so much felt as it would be with us in our long journeys.

Much allowance must be made for the modern English railway engineer. Whatever faults he may have seen in the passenger cars, he has been powerless to make any important change, except at an unjustifiable cost. Stephenson and his colleagues mounted the old stage-coach body on car wheels, which became the type of the passenger cars; and coal wagons that were then in use in the collieries were put on the railroad, and became the type of freight cars; and before the conservative English character thought that they ought to be improved, and should be changed, the trunk lines had been built, adapted to this narrow and low type of rolling stock; to have made it wider and higher later would have required the removing and reconstruction of the masonry platforms, the raising and widening of bridges and tunnels—in fact, almost a reconstruction of the road. This will prevent the use of our high, wide and pleasant cars. It is not fair to blame the modern English engineer for continuing the use of this description of cars, which he cannot change at any justifiable expense.

Owing to the extent of the United States and the apparent isolation of our early railroads, or separation from each other (at the beginning we did not think as much of continuous connections, and not breaking bulk as we do now), our engineers were comparatively free to invent and adopt the best system according to their judgment, and later, when it became necessary to consolidate, the fittest has survived in our present system. Perhaps, owing to our bad roads, the American stage-coach did not come up to the American engineer's idea of luxurious or comfortable traveling; hence he kept on improving, and developed the Pullman hotel train as run between New York and St. Louis or Chicago, while on the good English roads this stage-coach came up to Stephenson's ideas of comfort and luxury.

Engineering, Locating, etc.

It is evident that the English engineer was not obliged to study economy so closely as the American in locating or constructing; judging by the work, the former has always had at his disposal abundance of time and money. The roads built by him could not have been built, as some of our American roads have been, at the rate of over five miles per day; or, as some were done, by laying the rails first and afterwards

building the road. The English engineer seems to have the idea that the railroad should be built as near a straight line and level as possible, and is willing to spend a much larger amount to approach this ideal perfection than the American engineer is. Consequently the English railroad is located much straighter, with easier curves and grades than the American location would have been over the same ground, but at greatly increased cost in money and time required for construction. Recently English engineers have introduced on their new roads curves and grades that would have been considered impracticable by Stephenson and his contemporaries.

In locating railroads in England, the engineer derives much assistance from the maps of the Ordnance Survey, which are so perfect that the route and approximate location can be determined before going on the ground in all the counties; while some counties are so perfectly contoured that a location and profile could be made without a survey, which would be correct enough for preliminary purposes.

Special Acts of Parliament must be obtained for every new road or branch. Before this can be applied for, much less obtained, there must be a very exact profile and map filed, showing everything within three hundred and thirty feet on each side of the proposed line. If Parliament should grant the application, only a very small deviation in grade or alignment is allowed from the plan and profile filed originally. These plans must be very correct. There is a story told that an application, strongly backed, was thrown out because a pig-sty had been omitted on the map filed. This application before Parliament is generally bitterly opposed by rival roads and interests, and the fight becomes long and expensive, each side employing prominent engineers and lawyers. It is said that in a recent case before Parliament, a prominent American engineer received 2 000 guineas ($10 500) for his testimony, which he gave in two and a half hours.

The following table shows the money that has been expended in promoting or opposing Bills before Parliament for eleven years, from 1871 to 1882 inclusive:

Municipal bodies	£1 289 757
Railway, Gas and Water Companies	4 664 874
Canal and Tramway Companies	416 043
Harbor and Dock Companies	360 574
Total	£6 731 248
At $5 per pound sterling	$33 656 240

About $3 000 000 annually, and this in dull times, after all the principal improvements had been completed or commenced. This amount is admitted to have been paid; probably much more was paid and charged to other accounts. Of this amount, £380 160 went in "House fees" paid to the House of Commons; say the fees paid to the House of Lords would be as much, there still remains £6 000 000 admitted to have been paid. This parliamentary business is very good for the English engineers; probably many of them make as much, or more, engineering in the Houses of Parliament as they do in the field. As a proof of the importance of this parliamentary engineering to the profession, nearly all the prominent engineers have their offices in Westminster, near the Houses of Parliament, and not in the city near the money center as with us.

The right of way is easily and quickly obtained after Parliament grants the right to build the road. In this grant is inserted what is called "Clauses Consolidation Acts." This provides that, in case the necessary land cannot be obtained on satisfactory terms by private purchase, application may be made to certain judges or magistrates named, who order the land in dispute to be appraised and the amount of this appraisement to be deposited in the Bank of England. As soon as proof is produced of this deposit, possession is given of the land. The money deposited is held, subject to the final settlement, by the Courts or otherwise, of the money to be paid for the land.

England offers no very serious engineering difficulties against railroad construction. But few lines attain the elevation of 900 feet above the sea; they have no high backbone divides to be crossed as we have. Probably the whole of England will compare in this respect with the United States, after excluding the strictly Prairie States. In round numbers, half of our railroads are constructed in a country offering no greater advantage than the United Kingdom for cheap construction.

English railroad men—it may be said foreigners generally—have a very erroneous idea about the physical geography of the United States. Nearly all believe that the American roads are built at a comparatively small cost, because the whole country is a level prairie, with nothing to do but to lay the ties and rails on the prairie sod. An old officer in the railroad department of the English Board of Trade was very much astonished when he was told that all our American roads are not built on the level prairie.

Cost of Land and Right of Way.

This is a large item in the cost of most English railroads. It is impossible to say how much, as no reliable data could be obtained. Some say it will average twenty thousand dollars per mile; this is undoubtedly too high. Cases have been mentioned where the price paid has been so large, that in our plain American language it would be called blackmail, but in the more polite English it is called land damages.

Construction.

This is generally much superior, stronger, more substantial, and a great deal more costly on the English roads than on ours. This difference is much greater on new roads. Our system of rushing the road through, and completing it afterwards at leisure times, is not practiced there. The Board of Trade impartially obliges every road, large or small, to be made at first up to its standard of perfection; consequently all English roads are well made at first, and kept up—all are first-class; if one falls below the standard, it is closed. This would be rather hard on some of our roads.

Wooden culverts or trestle-work are unknown in England. Brick viaducts are generally used where we would use trestle-work, or embankments in shallow depressions. Bridges are almost always built of brick, stone or iron; wooden ones are very rare.

Platforms and stations are generally built of masonry; some few are built of wood.

Embankments in England are generally made flatter, or with more slope than ours. Cuts are generally allowed to take their natural slope, though much more attention is paid in England to foot and side drains than with us.

The ballast in England is generally sandy gravel; there is no such road-bed as that on the Pennsylvania or Reading Railroad. The ties are sawn, generally over 10 inches wide, 6 inches thick and 9 feet long, and placed about 34 inches from center to center; at joints they are somewhat closer.

The rails generally used are the double-headed or reversible pattern, running up to 87 pounds per yard, with 52-pound chairs, on some of the railways running into London. The rails are secured into the chairs by wooden wedges or blocks. Some few flat or flanged bottom, called Vignolles or Colonial rails, are used, but they do not seem to be popular. Rails are generally 30 feet long, and made of Bessemer steel.

The English engineer is much favored by his mild winters. The frost is not sufficiently severe or penetrating to affect the stability of his road-bed or superstructure. The snows are comparatively very light, and would give no trouble, provided the railroad had the most simple ploughs in use on our Northern railroads for removing it.

In England there are no street, and very few ordinary road-crossings on a level, and these few are always provided with a gate and watchmen. The Board of Trade is very stringent in this respect, and we should imitate them by passing a law obliging all railroads in populous districts to do the same.

The ties are buried deep, apparently to make the road-bed as solid as possible, and on the end of the ties the ballast is heaped up to the level of the top of the rails. On some roads the chairs are bolted to the ties by screw-bolts, with the nuts under the tie.

As a sample of how costly the English railroads are built, the author contemplated building a new station, and got from the superintendent of the railroad the cost of building it, which was $11 000. There was no grading necessary, all that was required was simply two platforms with shed roofs, connected by an overhead bridge, 14½ feet headway, with two stairways leading from the platforms to the bridge.

Mr. Webb, the able Locomotive Superintendent of the London and North Western Railway, has recently introduced a steel sleeper that gives great satisfaction and promise. It is 9 feet long, weighing 124 pounds; near each end the sleeper is punched with six holes for the chairs. The chairs, weighing 14 pounds, are rolled from the waste ends of the steel rails, etc., then stamped into shape and holes punched. A steel lining plate, ¼ inch wide, is placed between the chair and sleeper. To prevent noise and working loose, brown paper, soaked in tar, is placed between the chair and plate, and plate and sleeper; then the whole is riveted together with steel rivets. This makes a very firm and solid road-bed, without any possibility of the rails spreading. The total weight of the sleeper, chairs and plates complete is 174 pounds, and at the present price of steel should not cost in the United States over 1½ cents per pound, or $2.61 for each sleeper. Mr. Webb says the total cost of labor on this sleeper is 5½d. = 11 cents. Total cost of sleeper at present prices in England, 8s. 3d., say $2.00. The rails are fastened, or rather secured, into the chairs by wooden wedges, as is usual on the English roads. The life of this sleeper is as yet an unknown quantity; rust is probably

the only thing that will destroy it. If the constant jar would not break or detach the asphalt coating used so successfully on water-pipes in California; or the coating of Dr. Angus Smith, used in many places satisfactorily on water-pipes; provided either of these coatings could be kept on intact, the life would be very long and there would be no doubt as to the economy of adopting it. The London and North Western Railway have put down five miles of these ties in order to test them thoroughly.

Some of these have been in use for five years, and as yet show no signs of weakness or decay. The coating still remains intact, notwithstanding the five years' service, with the frequent jar of heavy passing trains. Mr. Webb anticipates no trouble from rust. He has taken up water-pipes in good condition that have been buried for sixteen years, which were coated, previously to being buried, by the coating of Dr. Angus Smith. Mr. Webb's sleeper or tie will probably come into general use, as it possesses the maximum strength with the minimum weight and cost.

Locomotives.

For fast passenger trains, the engine generally has two large drivers about 7 feet 8 inches diameter; outside connected cylinders, horizontal; with bogie truck in front, and two trailing wheels. For slower passenger and freight trains generally four drivers; inside cylinders, outside connected; some with bogie trucks. Some freight engines have six drivers, outside connected, with inside inclined cylinders; all weight of engine on drivers. Many new engines are now built with four driving wheels; outside cylinders and connections; cylinders generally inclined.

The cab has finally been introduced upon most of the English engines, though there are still many without it. But it is not such luxurious quarters as the American engine-driver has; it is simply a small iron hood with small and fixed glass bull's-eyes, or windows that cannot be opened as with us, consequently they are very warm, and many of the engine-drivers object to the cab for that reason. It seems strange that no one has thought of our plan of having the windows large and on hinges, to be opened or closed at pleasure.

The bogie truck and outside connections, avoiding the crank axle, have had a hard and severe fight for favor on the English locomotive, but finally their merit has been recognized and many of the new locomotives are built on this system. Chairman Moon, at the half-yearly meeting of the London and North Western Railway Company, August,

1884, said: "Nothing has pleased me more lately than to see the old-fashioned crank axle done away with. We have had no accidents with it, but we are going to have something better than the crank axle."

It may not be amiss to state that on the Continent, engines with inside cylinders and connections have been almost entirely superseded by those with outside cylinders and connections.

Consumption of Fuel.

Many limited tests and experiments have been reported, but all, in my judgment, give much too high efficiency for the locomotive as a power producer, except on a limited experimental scale. Mr. William Strudley, M. Inst. C. E., Locomotive Superintendent of the London, Brighton and South Coast Railway, recently read a paper before the Institution of Civil Engineers. He stated that, in a locomotive of his design, one pound of coal will evaporate 12.95 pounds of water, and will convey one ton of train-load 13½ miles at an average speed of 43.38 miles, the rate of consumption being 2.03 pounds of coal per horse-power per hour. The French engineers have reported recently 2.48 to 3.01 pounds of coal per horse-power per hour. In my judgment all these estimates are entirely too favorable; if not, the locomotive is not as expensive a fuel-burner as is generally thought.

Soft coal is universally used, except on the railroads in London, where a semi-anthracite is burned. This is used on account of giving off very little smoke, which would be very objectionable in the city.

Passenger Cars.

The extreme width of a passenger car that can be run on the average four feet eight and a half-inch gauge English railroad, is 8 feet 10 inches from extremity to extremity of eaves; this makes the cars too narrow to adopt the American construction of central aisle with double seats on each side. If it should be desired to adopt this system of central aisle, it would be necessary to reduce the length of the seats on one side from the length for two persons to that for one, as in some of our 3-feet gauge cars.

The Pullman cars that are run in England have been reduced in width from 10 feet to 8 feet 10 inches from eaves to eaves, and in height 13 inches. They are constructed on the same plan as generally with us, with a central aisle; the seats are too short to seat two persons

comfortably. They resemble very closely the Pullman cars made for our 3-feet gauge railroads.

For a short distance, and until the close confinement becomes tedious, nothing can be more comfortable and luxurious than the first-class English cars in pleasant weather. The absence of all toilet conveniences is a great inconvenience on long journeys.

On most railroads in England there is still no way of communicating between passenger, conductor and engine-driver, or from one car to another while the train is running. The London, Chatham and Dover road uses a bell-rope passing along the top of the floor, and the Great Eastern uses the bell-rope passing along the outside of the cars just under the eaves; this can easily be reached by opening the car window and reaching up.

The English passenger car is cold in cold weather, and hot and close in warm weather. If one is seated facing the engine he is in a gale, if seated on the front seat, is in a corner where no air can reach him. It does not run as smooth as the American; it is more difficult to read, and impossible to write in them.

On the London and North Western Railway, passenger trains with a seating capacity for 95 first-class, 60 second-class and 150 third-class—total, 305 passengers—and vans for their baggage, weigh 151.8 tons, or half a ton for each passenger.

Freight Cars and Trains.

The English freight car is about 15.5 feet long, with four wheels. About 90 per cent. of them are open cars, with sides and ends about 3 feet high. About one-fourth of each side at the middle of the car is on hinges, so as to open and facilitate unloading. About one-tenth of the cars are covered, corresponding to our box cars; these are generally covered with tarpaulins tied on; the roofs seem to be used more for safety or protection against robbery than for protection against rain. The open cars are always covered by a tarpaulin tied on, except when loaded with coal or other load that cannot be injured by the weather. These cars are coupled by heavy chains, and each car has about 1 foot of slack, consequently in starting the load comes very slowly upon the engine.

Only about 20 per cent. of the cars have brakes, and these are so placed that they cannot be manipulated while the train is in motion. In the rear of the train is a caboose with a brake; this, with those on the

engine and tender, is the only braking force that can be applied while the train is in motion. In long trains, or on steep grades, additional brake vans are attached, which make, on some roads, quite an important additional expense in the salary of the brakemen, and hauling up grade the dead weight of the brake vans, which, in order to be effective, are made very heavy.

These trains run generally about 25 miles an hour. The only hands employed on them are the engineer (called there the driver) and fireman on the engine, and the conductor in the tail caboose, with no means of communication with each other except by signs—the useful bell-rope not being used. One would think it would be impossible to avoid many serious accidents for the want of this communication between the conductor and the engine-driver, but they seem very fortunate in avoiding them. There is not much danger of a train breaking in two.

The average English freight car, or, as called in England, "goods wagon," carries a load of 8 tons, and weighs 5 tons, being 1.6 to 1; while with the American box freight car, carrying 50 000 pounds, and weighing 23 000 pounds, the proportion is 2.13 to 1.

Without any positive data before him, the author would judge that the first cost for the same amount of carrying tonnage is about the same in the two countries, but the cost of maintenance is probably in favor of the American car, as for the same carrying tonnage there are fewer pieces to be looked after and kept in order.

All freight cars are made of wood. Self-dumping or unloading cars are not in general use. Coal is generally unloaded by being shoveled into a wheelbarrow and wheeled to the pile; in some large yards it is unloaded by shovels, falling by gravity to the pile; in others, through a trap-door in the bottom of the car.

The English freight trains run at much greater speed than those of the United States, probably about 25 miles per hour. They do not average one-half of tons freight that the American trains do, while, owing to the superior construction of the roads, the comparatively straight lines and easy grades, the average train tonnage should be much larger —if not double.

The railway companies complain that the shippers require prompt delivery of goods. This could probably be done by the proper development of the express system, as practiced by the different railroads in the United States. This assures prompt delivery of fast freight, while

the slow freight comes at a much slower and profitable rate of speed to the railroad companies. By the reduction of the speed of the freight trains and increasing their load on the railroads of the United Kingdom, the cost of transportation could probably be largely reduced.

Passenger Fares.

The Midland Railway, which penetrates very thoroughly the Midland Counties of England and Southern Counties of Scotland, several years ago dispensed with the second-class carriages, and adopted the following rates:

First-class, per mile 1½d., and 5 per cent. Government tax added = 3⅛ cents. Third-class, per mile 1d. = 2 cents.

During the year 1883 the returns of the Midland Railway show that 94 per cent. of the total passenger traffic was third-class, which yielded 90 per cent. of the gross passenger receipts. Railways that have been brought into direct competition with it have been obliged to adopt the same rates, making their second-class fares about 2¾ cents per mile.

Where there is no competition, the following may be considered the average rates:

	Average per Mile.
First-class	4½ to 5 cents.
Second-class	3 to 3½ "
Third-class	2 "

The Great Northern Railway, one of the leading railroads of England, has recently decided to imitate the Midland, and only run first and third-class carriages on some of their lines. It is only a matter of time when all other roads will do the same.

It is only a matter of time when railways will dispense with or modify the first-class travel so as to make it pay; it is safe to say that no road in England now makes any money from its first-class travel. As long as they continue to sell first-class tickets, they must provide first-class carriages, as it is impossible to say beforehand what the transient travel will be; the consequence is, nearly every train has a large excess of empty first-class carriages, in order to accommodate all travel that may offer, as no one holding a first-class ticket would consent to take a seat in a second or third-class carriage. Moreover, the first-class passengers, even if the carriage were always filled, do not pay the road as well as the second or third-class. A compartment in a first-class carriage contains about 48 square feet, measured from

the outside of the car to the center of the dividing partition, and will seat eight persons. This gives 6 square feet to each person, which, at the price charged by the Midland Railway per mile, makes 6 divided by $3\frac{1}{2}$, say half of 1 cent per mile for each square foot of the floor of the car. The second and third-class apartments contain 40 and 35 square feet, and will carry 10 passengers. Taking the average second and third-class fares, this will give about two-thirds of 1 cent per mile for each square foot of floor surface. The dead load being about in the same proportion as the floor surface, it is clear that the first-class passenger is the most unproductive; moreover, the first cost of the first-class compartment and its maintenance is probably 20 per cent. greater than the second-class compartment, and 40 per cent. greater than the third-class.

All roads in the United Kingdom, except the Midland, virtually run six classes of carriages, viz., ordinary and smoking first-class, ordinary and smoking second-class, ordinary and smoking third-class. As it is impossible to fill all of these uniformly, there must necessarily be great loss of room and unnecessary haulage of dead weight.

The railroads running into London run what are called workmen's trains; by these trains working men or working women can go to and from their homes and work (say up to 15 miles) for 24 cents a week; this allows them to live in the suburbs where it is more healthy and rents are cheaper; weekly tickets are sold for the above amount, good for special designated trains in the morning and evening. Our roads should imitate the English roads in this respect.

Brakes.

In passenger trains there is generally one car in every three or four that have brakes. The vacuum brake, after a long and hard fight, has gained the victory, and is coming into general use on passenger cars. On very steep grades, when the brakes on the engine and the few brake cars would not be sufficient to hold the train in case of accident to the air brake, platform cars heavily loaded with cast-iron, with powerful brakes, are attached to each train going up or down, in order to hold the train in case of accident to the vacuum brakes; an expensive inheritance from the narrow stage-coach model, which could be avoided if the brakemen could go through a central aisle from car to car, and apply the brakes on each car as with us.

The freight trains have no brakes that can be used while the train is in motion, except those on the engine and caboose.

Our system of brakes could not be applied to freight trains there, as, owing to the lowness of the tunnels and bridges—another expensive inheritance from the old stage-coach model—brakemen could not walk over and along the top of the cars, and apply the brakes as with us. The only brake that could be adopted with safety would be one that could be worked from the engine or caboose, or buffer or compression brakes.

Block System.

It is astonishing to see the blind faith the English engine-driver places in his block signals. In dense fogs, where he cannot see one hundred feet ahead, or dark nights, when his vision is also very limited, for his head light is only an ordinary lantern, useless for illuminating the track and only used as a signal, the same as the tail light, or frequently where he has both the dark night and dense fog to run through, yet he runs at full speed and generally on schedule time, feeling sure that he is perfectly safe because his block signals have told him so, and they cannot make a mistake or lie.

The underground railroad in London and the London suburban railroads afford a fine illustration of this system. Notwithstanding the proverbial London fogs, these roads run their trains, probably several thousand daily, upon schedule time, and with a headway varying from three to three and a half minutes. And this is done without accident or delay.

If it had been in use, the accident two or three years ago in the Harlem Tunnel and also that at Spuyten Duyvil, in which Senator Wagner and others were killed, could not have occurred.

The English Government, through the Board of Trade, obliges all English railroads to adopt the block system, and run their trains by it, and we should follow their example. The officers of a railroad should be tried and convicted of premeditated murder in all cases of fatal accidents on their road that could have been avoided by the block system. It is no excuse for them to say it does not answer, for it does answer perfectly in England and other European countries; the officers controlling the roads ought to be made to feel that it is cheaper and safer to properly equip the road by using the block system, than to kill or maim passengers by not doing so.

Speed.

English freight trains run much faster than ours; probably averaging twenty-five miles an hour.

The passenger trains are generally faster, especially for short runs; but, owing to the small size of the island, there are no such trains as the New York and Chicago or the New York and St. Louis Express.

Table No. 1 gives the fastest trains on the principal roads of each country.

Baggage Checks.

These are still unknown in England. Some of the large railroads will receive baggage and deliver it at a given destination for a charge of sixpence for each package, provided the places at which it is received and delivered are both on their line, and are large places, such as London, Liverpool, Manchester, etc. There is nothing like our through check system over many distinct lines. This would be a very great convenience to those who have to travel in England, as the care of the baggage requires constant attention to see that some one else does not take it. It is placed in the baggage car without any distinctive mark of ownership, and subject to the call of the first claimant. Yet one seldom hears of its loss. This speaks well for the honesty of the English people.

The railroad managers and superintendents seem averse to making any change, even if it would promote the comfort of the traveler, and thereby induce increased travel. Some oppose the introduction of the check system for fear that the road would lose by being held responsible for lost baggage. In vain the author quoted to them the experience of the Pennsylvania Railroad, which handled in 1883 the baggage of over 23 000 000 passengers, amounting to nearly 2 000 000 pieces, and only had to pay in the whole year for lost and damaged baggage $1 262.03.

The English railroads could save a large expense by adopting the American baggage check system, as it would enable them to dispense with a large force of porters now employed in loading and unloading the baggage of passengers. Eighty per cent. of this labor would be done by the baggage express or transfer companies free of charge to the railroads.

TABLE No. 1.

Table showing the Fastest Speed made by Regular Trains on English and American Railroads between Terminal or Important Stations—Time-tables, June, 1885.

Names of Railroads.	From	To	Distance in Miles.	Time, Including Stoppages.		Average Miles per Hour, Including Stoppages.
				Hours.	Minutes.	
English Railroads.						
London and North Western	London	Liverpool	201.75	4	30	44.8
" "	"	Glasgow	406	10	30	40.6
" "	"	Edinburgh	401	9	55	40.4
" " (Wild Irishman)	"	Holyhead	264	6	5	43.4
Great Northern	"	Glasgow	444	10	20	43.0
" (Flying Scotchman)	"	York	188.25	3	55	48.1
"	"	Edinburgh	397	9	55	44.1
Great Western	"	Swansea	216	6	55	36.0
" (Flying Dutchman)	"	Bristol	118.5	2	36	45.6
London, Brighton and South Coast	"	Brighton	50	1	5	46.2
" " "	"	Dover	76.5	1	40	45.9
Midland	"	Nottingham	125	2	30	50.0
London, Chatham and Dover	"	Dover	77.75	1	48	43.2
American Railroads.						
New York, New Haven and Hartford	Boston	New York	234	6	0	39.0
New York and New England	"	"	227	6	0	37.8
Pennsylvania	Jersey City	Philadelphia	89	1	57	45.6
"	"	Pittsburgh	443	11	15	39.4
"	"	Chicago	911	24	45	36.8
New York Central and Hudson River	New York	Albany	143	3	15	44.0
" " "	"	Buffalo	441	10	45	41.0
" " "	"	Chicago	980	25	0	39.2
Central of New Jersey	Jersey City	Philadelphia	89	1	52	47.7
Baltimore and Ohio	Baltimore	Washington	40	0	45	53.0

Comfort in Traveling.

In the opinion of the writer this is not so great in England as with us. In addition to the trouble with the baggage, there is the discomfort of riding in winter in a cold car, obliging one to wrap up even for a short journey as if he was going to the Arctic Regions. No attempt at warming has been made, except by hot-water pans to the feet. Another serious inconvenience is the difficulty of getting information while *en route*, owing to the want of communication between the different compartments. It is impossible to ask questions of the train hands whilst in motion, and during the few limited stops at stations the employees are busy with their duties and endeavoring to answer the questions of many others in similar want of information. The writer once traveled from London to Gloucester, 114 miles, without being able to find out the destination of the car he was in.

Unless one is bound for a terminal station, he is obliged to be constantly on the alert, or he is apt to be carried beyond his destination. The train hands generally call out the names of the stations, but in so disguised and mispronounced a manner that they are unrecognizable. At stations there is often no one previous to the arrival of the train except the gatekeeper, who is generally a long distance off, up or down a flight of stairs, that can give any information about the trains.

The Board of Trade requires that a sign, with the name, must be placed at each station; but as station advertising is carried to a great extent throughout England, it is very difficult to recognise the station sign from the hundreds of advertisements, equally conspicuous, of "Pears' Soap," "Lorne Whiskey," "Coleman's Mustard," etc., surrounding the name of the station.

Ton and Passenger Mileage.

No return of these all-important items for comparison is made in England. The author, after careful inquiry and investigation, decided that the average freight charge on all freight moved in the United Kingdom was about 1¼ pence, or 2.5 cents per mile per ton; but, in order to be conservative, he has taken the average charge at 1 penny, or 2 cents per ton per mile.

In order to get the ton mileage, the receipts from freight, as reported by the companies, in pounds sterling has been multiplied by 240.

The author thinks, as do most of the persons with whom he has con-

sulted, that the average freight charge of $0.02 or 1 penny per mile per ton is too low by at least one-fourth; $0.025 or 1¼ pence would be nearer the correct amount. This more than makes up the difference between the English ton of 2 240 pounds and the American ton of 2 000 pounds. The English tonnage returns are all given in tons of 2 240 pounds, while the American returns are all in tons of 2 000 pounds.

Mr. J. S. Jeans, the able Secretary of the Iron and Steel Institute of England, who is very justly regarded as one of the most able statisticians of Great Britain, estimates that the average freight charge per mile on minerals is $0.015 or .75 pence per ton per mile. This would make the preceding average charge on all classes of freight of $0.02 or 1 penny per mile much too small.

The passenger mileage has been obtained by dividing the reported total receipts from ordinary passengers by the total number transported. This gives the average amount received from each passenger. Divide this by $0.0233*, gives the average mileage traveled by each passenger. Multiplying this by the number transported, gives the total mileage of ordinary passengers. In order to save labor and time, many of the following calculations for passenger mileage were made by multiplying the total reported receipts in pounds sterling from ordinary passengers by 206, the estimated average number of miles that can be traveled for each pound paid.

The mileage of the holders of season tickets has been ascertained by comparing the schedule lists of prices for season tickets as issued by the different railroads leading into London, with the distances; the average price is found to be about $0.009 or .45 pence per mile—say, $0.008 or .4 pence. This makes 600 miles as the average mileage traveled for each pound sterling reported as received from the sale of season tickets. Multiplying this reported amount by 600, gives the total mileage traveled by the holders of season tickets. Adding this to the total mileage of the ordinary passengers, gives the total mileage of all passengers.

It is very unfortunate for comparison that the English railroads do not report the passenger or ton mileage. The preceding figures have been adopted after careful inquiry and examination, and can be considered approximatively reliable until changed by official data from the companies.

*One and one-sixth pence being the estimated charge per passenger per mile on all the English railroads, first, second and third-class included, exclusive of season tickets.

The passenger and ton mileage for the American railroads have been taken from the official reports of the companies or from the reports of the Railroad Commissioners of the different States, and are reliable.

ACCIDENTS.

TABLE No. 2.

Passengers Killed and Injured from causes beyond their own control on all the Railroads of the United Kingdom, and those of the States of New York and Massachusetts in 1884.

	Total Length of Line Operated.	Total Mileage. Train.	Passenger.	Ton.	Killed.	Injured.
United Kingdom	18 864	272 803 220	6 042 659 990	9 040 942 080	31	864
New York	7 298	85 918 677	1 729 653 620	9 322 518 571	10	124
Massachusetts	2 852	32 304 333	1 007 136 376	1 229 368 472	2	42
In 1 000 000 000 passengers transported 1 mile	United Kingdom				5.15	143
	New York				5.78	70
	Massachusetts				2.00	42
In 1 000 000 000 tons transported 1 mile	United Kingdom				3.44	96
	New York				1.11	14
	Massachusetts				1.63	34
					Miles.	
The average number of miles a passenger can travel without being killed	United Kingdom				194 892 255	
	New York				172 965 362	
	Massachusetts				503 568 188	
The average number of miles a passenger can travel without being injured	United Kingdom				6 992 662	
	New York				13 940 754	
	Massachusetts				23 955 630	

This table shows the comparative magnitude of the American railroad system. One single State has over 40 per cent. of the miles of railroad that the United Kingdom has, and transports more freight and nearly 30 per cent. of as many passengers.

The Board of Trade of England publish annually very complete returns of accidents on all their railroads. Unfortunately there are no such returns available for all the American roads.

The Railroad Commissioners of Massachusetts and New York make very complete returns for their States, which can be taken as a fair sample of the old and completed roads of the United States. The new

roads of the West will show a much higher percentage of accidents, owing to being open to traffic generally during construction—and being generally not so well built.

Table No. 2 was constructed from the preceding data, and the reports of the English Board of Trade, and of the Railroad Commissioners of New York and Massachusetts.

Cost of Railroads.

A casual observation shows that the English railroads cost much more than the American in their construction. It is evident that the English engineer always had in view that there was behind him plenty of money, and abundance of time to spend it in. Apparently there was no necessity for studying an economical alignment—heavy cuts and fillings or long bridges and tunnels, did not appear to frighten him at all; his ideal line was a straight line and dead level, and he kept as near this as circumstances would permit. This expensive construction, with apparently straight lines and easy grades, good road-bed, permanent brick buildings, and brick, stone or iron bridges, should make the annual expenses much lighter than on our roads, where perishable materials, for economical reasons, enter so largely into our buildings, bridges, etc., requiring frequent renewals.

The question naturally arises: Do the working results compensate for this great additional cost of the English over our railroads? In other words, "Does it pay?" To ascertain this, we must analyze the working expenses of each system; this is not an easy undertaking, as the published reports of the two countries do not correspond in form or outline. Unfortunately, the returns of the English railroads made to the Board of Trade, and the annual reports of the different companies, are not as thorough as one would wish for comparison. They do not give the quantity or cost of coal consumed, or separate the cost of freight or passenger traffic, or give the total or average ton and passenger mileage.

In order that you may investigate for yourselves, the problem whether it pays or not to spend so much money in constructing railroads, the Tables from No. 3 to No. 17 have been made, showing the operating expenses of all the railroads in the aggregate, and also of a few of the principal roads of each country. The principal, if not the only, advantage of expensive over cheap construction, is to reduce the operating expenses; to ascertain to what extent this has been done, in the

following accounts of the operating expenses all items of expense that are in any way affected by good or bad construction, have been added together, and the percentage calculated to the whole operating expenses. This percentage will naturally be larger in cheaply constructed roads with sharp curves and steep grades, and built with perishable materials, than in the expensively built roads in which money has been freely spent to reduce curves and grades, and in the use of more durable materials.

Mr. Jeans, the able statistician of England, makes this percentage 59.66 for all the American railroads. This has been taken by the author as the average standard of all the railroads in the United States; to this standard the English roads have been compared. The percentage of the individual American roads has been worked out from data contained in their annual reports, or from the reports of the Railroad Commissioners of the different States.

TABLE No. 3.

Comparative Proportion of Different Items of Operating Expenses, that are or are not Affected by Good or Bad Construction, on the Aggregate Railroads of the United Kingdom and United States for the year 1880.

1880.	United Kingdom.		United States.	
	Expenses that are Affected by good or bad Construction.	Expenses that are not Affected by Good or Bad Construction.	Expenses that are Affected by Good or Bad Construction.	Expenses that are not Affected by Good or Bad Construction.
Maintenance of Way	$17.70	.....	$25.31	
Locomotive Charges	24.40	.....	24.96	
Repairs of Carriages	8.40	.....	9.39	
Traffic Charges	.....	$30.00	.....	$25.31
Rates and Taxes	.....	5.00	.....	4.77
Government Duty	.....	2.40	.....	
Compensation—Passengers	.....	.70	.....	.39
Do. Goods	.....	.60	.....	.59
Legal and Parliamentary Expenses	.....	.90	.....	.70
Miscellaneous Expenses	.....	9.90	.....	8.58
Totals	$50.50	$49.50	$59.66	$40.34

This, and the two following tables, have been taken from a report recently made by Mr. J. S. Jeans, Secretary of the Iron and Steel Institute of Great Britain. Mr. Jeans is justly regarded as very good authority, and one of the most capable statisticians in this branch in England.

TABLE No. 4.

Showing the Number of Persons Employed on the Railroads of the United Kingdom and the United States in 1880.

1880.	United Kingdom.	United States.
Engine-drivers and Firemen	27 441	67 231
Guards	12 064	12 419
Plate-layers (Track-layers)	26 156	
Pointsmen, etc. (Switchmen)	7 406	122 489
Laborers	42 212	
Other railway servants	120 437	216 818
Totals	235 716	418 957
Average employés per 1 000 miles of goods and passenger trains	0.89	1.08
Average employés per mile of line open	13.	4.8
Average employés per 1 000 tons of traffic	0.9	0.7
Average employés per 1 000 passengers carried	0.38	0.65
Average employés per £1 000 earned	2.7	0.28

TABLE No. 5.

Showing the Rolling Stock used on Railroads of United Kingdom and United States in 1880.

1880.	United Kingdom.	United States.
Locomotives	14 469	17 412
Passenger Carriages	32 304	12 330
Freight Cars	446 333	375 312
Other Vehicles	12 024	84 613
Totals	505 130	489 667
Average vehicle per mile open	2.6	0.24
Average vehicle per 1 000 tons freight carried	0.5	0.59
Average vehicle per 1 000 passengers carried	1.3	0.53
Average vehicle per $5 000 capital	1.5	2.3

In the following Tables of the English Railroads, their percentage has been subtracted from 59.66, the average percentage of the American roads. This difference in the percentage, multiplied into the whole operating expenses, shows the gross saving by being well constructed. This divided by the number of miles operated, shows the annual saving per mile by the superior English over the inferior American construction. This amount capitalized at six per cent., shows the amount that the engineer would have been justified in expending per mile in construction over the cost of the inferior American system.

TABLE No. 6.

Operating Expenses of all the Railroads of the United Kingdom.

Year ending December 31st, 1883. Length of all Roads Operated, 18 681 Miles. Gauge, 4' 8½". Total cost, $3 777 808 395. Average Cost per Mile, $202 227.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way, works, etc.	$33 718 990	
Locomotive power, including stationary engines, etc.	46 506 335	
Repairs and renewals of passenger and freight cars	16 476 835	
Traffic expenses, freight and passenger	.....	$56 061 995
General charges	.....	8 118 705
Rates and taxes	.....	9 304 245
Government duty	.....	3 696 280
Compensation for personal injury	.....	1 235 160
Compensation for damage and loss of goods	.....	989 705
Legal and Parliamentary expenses	.....	1 865 925
Steamboat and harbor expenses	.....	6 624 780
Sundries	.....	2 269 410
Totals ($186 868 365)	$96 702 160	$90 166 205
Percentage	51.8	48.2
Percentage, average of all American railroads in 1880	59.7	
Percentage in favor of all the railroads of the United Kingdom	7.9	

Total saved by all the railroads of the United Kingdom, $186 868 365 × .079 = $14 762 600
Average saving per mile, $14 762 600 ÷ 18 681 = 790
Annual cost of $790 capitalized at 6 per cent = 13 167

TABLE No. 7.

Operating Expenses of the Great Eastern Railway of England.

Year ending December 31st, 1883. Length of Line Operated, 1 049 Miles. Gauge, 4' 8½".	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way	$1 481 740	
Locomotive power (including stationary engines)	2 575 085	
Repairs and renewals of passenger and freight cars	764 335	
Traffic expenses—freight and passenger	.....	$2 953 350
General charges	.....	387 070
Rates and taxes	.....	439 975
Government duty	.....	256 010
Compensation for personal injury	.....	84 795
Compensation for damage and loss of goods	.....	55 210
Legal and Parliamentary expenses	.....	76 020
Sundries	.....	810
Totals (*$9 074 400)	$4 821 160	$4 253 240*
Percentage	53.0	47.0
Percentage, average of all American railroads	59.7	40.3
Percentage in favor of Great Eastern Railway	6.7	

Total saved by Great Eastern Railway on operating 1 049 miles of railroad $9 074 400 × .067 = $607 985
Average saving per mile, $607 985 ÷ 1 049 = 580
Annual cost of $580 capitalized at 6 per cent. = 9 660

* The cost of operating the steamboats, etc. ($899 430), has been deducted as not properly belonging to the railroad service.

TABLE No. 8.

Operating Expenses of the Great Northern Railway of England.

Year ending December 31st, 1883. Length of Line Operated, 768 Miles. Gauge, 4′ 8½″.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way, works, etc.	$1 616 115	
Locomotive power (including stationary engines)	2 637 645	
Repairs and renewals of passenger and freight cars	842 130	
Traffic expenses—Freight and passenger		$3 326 745
General charges		493 245
Rates and taxes		533 745
Government duty		168 530
Compensation for personal injury		149 290
Compensation for damage and loss of goods		70 320
Legal and Parliamentary expenses		58 765
Steamboat, canal and harbor expenses		58 900
Sundries		91 610
Totals ($10 047 040)	$5 095 890	$4 951 150
Percentage	50.7	49.3
Percentage, average of all American railroads	59.7	40.3
Percentage in favor of Great Northern Railway	9.0	....

Total saved by Great Northern Railway on operating 768 miles of road $10 047 040 × .09 =	$904 234
Average saving per mile, $904 234 — 768 =	1 177
Annual cost of $1 177 capitalized at 6 per cent. =	19 617

TABLE No. 9.

Operating Expenses of the Midland Railway of England.

Year ending December 31st, 1883. Length of Line Operated, 1 793 Miles. Gauge, 4′ 8½″.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way, works, etc.	$3 276 645	
Locomotive power (including stationary engines).	5 342 330	
Repairs and renewals of passenger and freight cars	2 069 555	
Traffic expenses—Freight and passenger		$6 413 915
General charges		713 350
Rates and taxes		957 475
Government duty		264 810
Compensation for personal injury, etc.		27 530
Compensation for damage and loss of goods		115 485
Legal and Parliamentary expenses		87 205
Sundries		135 495
Steamboat and harbor expenses		9 590
Totals ($19 413 385)	$10 688 530	$8 724 855
Percentage	55.1	44.9
Percentage, average of all American roads	59.7	40.3
Percentage in favor of Midland Railway	4.6	....

Total saved by Midland Railway on operating 1 793 miles, $19 413 385 × .046 =	$893 016
Average saving per mile, $893 016 ÷ 1 793 =	647
Annual cost of $647 capitalized at 6 per cent. =	10 777

TABLE No. 10.

Operating expenses of the London and North Western Railway of England.

Year ending December 31st, 1883. Length of Line Operated, 1 793 Miles. Gauge, 4′ 8½″.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way, works, etc	$4 751 885	
Locomotive power (including stationary engines)	6 148 560	
Repairs and renewals of passenger and freight cars	1 670 015	
Traffic expenses—Freight and passenger		$9 787 585
General charges		1 236 520
Rates and taxes		1 178 800
Government duty		629 040
Compensation for personal injury, etc		149 665
Compensation for damage and loss of goods		191 140
Legal and Parliamentary expenses		275 485
Sundries		137 730
Totals (*$26 156 425)	$12 570 460	*$13 585 965
Percentage	48.0	52.0
Percentage, average of all American railroads in 1880	59.7	....
Percentage in favor of London and North Western Railway	11.7	....

Total saved by London and North Western Railway on operating 1 793 miles of railway, $26 904 425 × .117 =	$3 147 821
Average saving per mile, $3 147 821 ÷ 1 793 =	1 756
Annual cost of $1 756 capitalized at 6 per cent. =	29 267

TABLE No. 11.

Operating Expenses of the Great Western Railway of England.

Year ending December 31st, 1883. Length of Line Operated, 2 268 Miles. Gauge, 7′ and 4′ 8½″.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Maintenance of way, works, etc	$4 497 670	
Locomotive power, including stationary engines	4 822 400	
Repairs and renewals of passenger and freight cars	1 533 820	
Traffic expenses—Freight and passenger		$5 361 685
General charges		763 285
Rates and taxes		949 255
Government duty		599 095
Compensation for personal injury		39 585
Compensation for damage and loss of goods		153 680
Legal and Parliamentary expenses		258 035
Sundries		210
Totals († $18 978 720)	$10 853 890	†$8 124 830
Percentage	57.2	42.8
Percentage, average of all American roads	59.7	40.3
Percentage in favor of Great Western Railway	2.5	....

Total saved by Great Western Railway on operating 2 268 miles of railroad, $18 978 720 × .025 =	$474 468
Average saving per mile, $474 468 ÷ 2 268 =	209
Annual cost of $209 capitalized at 6 per cent =	3 483

* The cost of operating the steamboats, etc. ($748 030), has been deducted, as not properly belonging to the cost of operating the railroad.

† The cost of operating the steamboats, etc., $942 515, is not included, as not properly belonging to the cost of operating the railroad.

TABLE No. 12.

Operating Expenses of the Baltimore and Ohio Railroad.

Year ending September 30th, 1883. Length of Line Operated, 553 Miles. Gauge, 4' 8½''.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
General expenses		$200 733.89
Losses by accidents, etc	$56 609.68	
Repairs of railroad	796 575.85	
Repairs of water stations	13 039.13	
Repairs and construction of depots	186 663.06	
Repairs of bridges	58 574.31	
Repairs of telegraph line	76 781.20	
Repairs of stationary machinery	118 276.66	
Watching cuts		30 546.15
Watching tunnels		5 451.67
Watching bridges		18 736.58
Pumping water	28 217.28	
Repairs of locomotives	624 146.36	
Repairs of passenger cars	299 386.71	
Repairs of freight cars	709 028.67	
Cleaning engines and cars		92 415.36
Contingent expenses of the machinery department	7 186.12	
Fuel	319 143.36	
Preparing fuel and filling tenders	20 096.37	
Pay of tonnage engineers	228 365.55	
Pay of tonnage firemen	105 505.61	
Pay of passenger engineers	82 812.24	
Pay of passenger firemen	40 013.25	
Expenses of transportation, salaries of agents, telegraph operators, clerks, station expenses, etc		2 029 350.7
Totals ($6 147 665.80)	$3 770 431.41	$2 377 234.39
Percentage	61.3	38.7

TABLE No. 13.

Operating Expenses of Pennsylvania Railroad. No. 1, Pennsylvania Division.

Main Line and Branches—Pittsburgh to Philadelphia.

Year ending December 31st, 1883. Gauge, 4′ 9″. Length of Road Operated, 1313.5 Miles.		Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Conducting Transportation		.....	$6 078 110.47
Motive Power		$5 237 379.22	
Maintenance of Cars		2 741 772.51	
Maintenance of Way		3 660 491.47	
General Expenses		.....	603 659.89
Totals	$18 321 413.56	$11 639 643.20	$6 681 770.36
Percentage		63.5	36.5

OPERATIONS.

Receipts from Freights	$24 778 190.15
Receipts from Passengers	5 989 625.35
Receipts from other Sources	1 249 997.79
Total Receipts from all Earnings	$32 017 813.29
Net Earnings from Operating	13 696 399.73
Total Mileage in this Division	1 313.53
Percentage of Operating Expenses to Earnings	57.22
Tonnage Moved	21 674 160
Tonnage Mileage	2 996 892 567
Average Ton Mileage	138.5
Average Earnings per Ton per Mile	$ .00819
Average Cost per Ton per Mile	.00477
Average Profit per Ton per Mile	.00342
Total Passengers Transported	10 697 474
Total Passenger Mileage	244 710 876
Average Passenger Mileage	$22\frac{9}{10}$ Miles.
Average Earnings from each Passenger per Mile	$ .02448
Average Cost of Transporting each Passenger 1 Mile	.01640
Average Profit per Passenger per Mile	.00808

TABLE No. 14.

Operating Expenses of Pennsylvania Railroad. No. 3, Philadelphia and Erie Railroad Division.

Year ending December 31st, 1883. Gauge, 4' 9". Length of Road Operated, 287.5 Miles.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Conducting Transportation	.....	$890 418.09
Motive Power	$762 468.99	
Maintenance of Cars	341 361.92	
Maintenance of Way	626 574.62	
Totals ($2 620 823.62)	$1 730 405.53	$890 418.09
Percentage	66.0	34.0

OPERATIONS.

Receipts from Freight	$3 352 417.79
Receipts from Passengers	667 742.66
Receipts from other Sources	88 682.71
Total Receipts from all Earnings	$4 108 843.16
Net Earnings from Operating	1 488 019.54
Percentage Operating Expenses to Earnings	63.8
Total Mileage in this Division	287.56
Tonnage Moved	5 154 935
Tonnage Mileage	520 249 716
Average Ton Mileage	100.9
Average Earnings per Ton per Mile	$ .00624
Average Cost per Ton per Mile	.00416
Average Profit per Ton per Mile	.00218
Total Passengers Transported	1 045 054
Total Passenger Mileage	21 908 174
Average Passenger Mileage	21 Miles.
Average Earnings from each Passenger per Mile	$ .03048
Average Cost of Transporting Each Passenger 1 Mile	.02075
Average Profit per Passenger per Mile	.00973

TABLE No. 15.

Operating Expenses of Pennsylvania Railroad. No. 2, New Jersey Division.

Year ending December 31st, 1883. Gauge, 4′ 9″. Length of Road Operated, 435 Miles.	Expenses that are affected by Good or Bad Construction.	Expenses that are not affected by Good or Bad Construction.
Conducting Transportation	.....	$5 433 260.35
Motive Power	$2 618 034.64	
Maintenance of Cars	622 251.12	
Maintenance of Way	1 736 301.22	
General Expenses	.....	103 491.37
Canal Expenses	.....	291 574.56
Totals—Exclusive of Canal Expenses..($10 804 913.26)	$4 976 586.98	$5 828 326.28
Percentage—Excluding Canal Expenses	46.0	54.

OPERATIONS.

Receipts from Freight	$8 269 943.20
Receipts from Passengers	5 542 247.22
Receipts from other Sources	1 144 405.23
Total Receipts from all Earnings	$14 956 595.65
Net Earnings from Operating	4 151 682.39
Total Mileage in this Division	435.07
Percentage of Operating Expenses to Earnings	72.95
Tonnage Moved	8 855 567
Tonnage Mileage	542 827 918
Average Ton Mileage	61.3
Average Earnings per Ton per Mile	$ .01471
Average Cost per Ton per Mile	.01167
Average Profit per Ton per Mile	.00304
Total Passengers Transported	11 339 330
Total Passenger Mileage	238 551 431
Average Passenger Mileage	21 Miles.
Average Earnings from each Passenger per Mile	$ .02074
Average Cost of Transporting each Passenger 1 Mile	.01571
Average Profit per Passenger per Mile	.00503

Few American engineers have the opportunity of seeing the English railroad reports. Tables 18, 19, 20 and 21 are exact reprints of all of that part of the semi-annual reports of the London and North Western Railway that relate to traffic for the years 1882, 1883 and 1884.

These will also enable those who are desirous, to make their own analysis of the data, as the author has done.

TABLE No. 16.

Railroads of the United Kingdom and of the United States Compared—1883.

Length of Lines Operated.	United Kingdom. Total.	United Kingdom. Per Mile Operated.	United States. Total.	United States. Per Mile Operated.	Remarks.
Single Track, miles	8 576				
Double or more Tracks	10 105				
Total miles of Line operated	18 681	.....	110 414	.....	About 23 per cent. of the total of the American roads is double track.
Capital.					
Shares and Stock	$2 944 990 635	$157 646	$3 708 060 583	$30 759	
Loan and Debenture Stock	832 817 760	44 581	.....	.....	
Bonds	.....	.....	3 455 040 383	28 650	The English roads subscribed £29-359 633 to other lines of railroads—this has been deducted, otherwise it would appear charged twice.
Floating Debts	.....	.....	332 370 345	.....	
Total	$3 777 808 395	$202 227	$7 495 471 311	$62 176	
Train Service.					
Total mileage of Passenger Trains, miles	138 176 940	.....	193 960 013	.....	Including New York Elevated.
Total mileage of Freight Trains, including coal, miles	127 983 253	.....	350 108 255		
Total mileage of Mixed Trains, miles	2 737 043	.....			
Total Train mileage, miles	268 897 236	.....	544 068 268		
Freight Traffic.					
Minerals moved (coal, etc.), tons	189 485 612				
General merchandise moved "	76 897 356				
Total tons of freight moved, tons	266 382 968	.....	400 453 439	3 627	
Total ton mileage	9 288 316 560	.....	44 064 923 445		
Average Train Load in tons	71	.....	126		
Freight receipts from Minerals (coal, etc.)	$81 277 105	.....	.....	.....	These items for United Kingdom are taken from Board of Trade Reports for 1883. In the original they do not add up correctly.
" Merchandise	106 242 435	.....	.....	.....	
" Live Stock	5 862 230	.....	.....	.....	
Total Freight Receipts	$193 506 595	$10 359	$549 756 695	$5 000	
Average Receipts from each ton moved	$0.70	.....	$1 373		
Average Freight Charge per ton per mile	.....	$0.02	.....	$0.0124	This charge is estimated for the United Kingdom.
Average mileage of each ton moved, miles	35	.....	110	.....	For the United Kingdom this is based on the average charge per mile.
Passenger Traffic.					
Total number First-class Passengers transported	36 387 877				
" " Second-class " "	66 096 784				
" " Third-class " "	581 233 476				
" " of holders of season tickets transported	632 050				
Total passengers of all classes transported	684 350 187	36 632	404 811 584	3 666	New York City Elevated Railways included.
Total passenger mileage	5 969 825 966	.....	8 817 684 503	.....	For the United Kingdom these estimates of mileage, etc., are based upon the estimate of $0.0233, being the average passenger fare per mile.
Average mileage of each passenger transported, miles	7.2	.....	22	.....	
Average number of passengers transported in each train per passenger train mile	.....	43	.....	45.5	
Receipts from First-class passengers	$18 350 265	$983			
" " Second-class "	16 648 720	891			
" " Third-class "	85 250 320	4 563			
" " Season Ticket "	8 462 955				
Total Receipts from passengers	$128 712 260	$6 890	$221 633 082	2 007	New York City Elevated Railways included.
Average Receipts from each passenger	$0.1881	.....	$0.5475	.....	do
Average mileage of each passenger, miles	.....	8	.....	22	
Average charge per mile for each passenger	.....	$0.0233	.....	0.0242	Estimated for United Kingdom.
Earnings.					
Earnings from Freight Traffic	$193 506 595	$10 359	$549 756 695	$5 000	
" " Passenger Traffic	128 712 260	6 890	221 633 082	2 007	
Total Earnings from Freight and Passenger Traffic	$322 218 855	$17 249	$771 389 777	$7 007	
" " " all sources	355 311 350	19 020	823 772 924	7 461	
Percentage of Gross Earnings to Investment, Capital and Bonds	9.05	.....	10.99	.....	This is on the inflated or watered stock of the American railroads.
Percentage of Net Earnings to Investment, Capital and Bonds	4.29	.....	4.49	.....	
Operating Expenses of all Railroads operated	$186 812 810	$10 003	$486 861 040	4 410	
Percentage of Operating Expenses to Receipts	53.0	.....	59.0		
Total Net Earnings	$168 468 540	$9 019	$336 911 884	$3 051	
Percentage of Net Earnings to Gross Receipts	47.0	.....	41.0		
Passenger Earnings per Passenger Train mile	.....	$0.925	.....	$1.14	
Freight " " Freight " "	.....	1.50	.....	1.56	
Rolling Stock.					
Total Engines	14 469	.775	23 823	.216	
Total Passenger Cars	32 304	1.73	17 899	.162	
Total Baggage, Mail and Express Cars	2 024	.108	5 918	.054	
Total Freight Cars	446 333	23.90	748 661	6.78	

TABLE No. 17.

Showing the Percentage of the Annual Operating Expenses that is affected by Good or Bad Construction, of some of the Principal Railroads of the United Kingdom and United States.

Name of Railroad.	Country.	Miles of Line Operated.	Average Cost as shown by the Books. Per Mile.	Percentage.		Average Charges Per Mile.		Average Saving per Mile by good Construction over average American Railroads.	This Annual Saving capitalized at Six per cent.	Remarks.
				Affected.	Not Affected.	Ton.	Passenger			This last column shows how much could have been spent advantageously in addition over the average cost of American Railroads.
		Miles.	Dollars.	Per cent.	Per cent.	Dollars.	Dollars.	Dollars.	Dollars.	
All the Railroads, average of....	United Kingdom	18 681	202 227	51.8	48.2	0.02	0.0233	790	13 167	
London and South Western.....	England..	721	.....	49.7	50.3	.....	.....	1 142	19 033	
Great Eastern....	"	1 049	.....	53.0	47.0	.....	.....	580	9 660	
Great Northern.....	"	768	.....	50.7	49.3	.....	.....	1 117	19 617	
Midland.....	"	1 381	.....	55.1	44.9	.....	.....	647	10 777	
London and North Western.....	"	1 793	.....	48.0	52.0	.....	.....	1 756	29 267	
Great Western.....	"	2 268	.....	57.2	42.8	.....	.....	209	3 483	
London, Chatham and Dover.....	"	160	744 970	51.5	48.5	.....	.....	1 496	24 933	
Caledonian.....	Scotland .	897	220 823	54.5	45.5	.....	.....	466	7 766	
North British.....	"	1 006	161 355	56.4	43.6	.....	.....	216	3 600	
Midland Great Western of.....	Ireland . .	425	69 250	62.0	38.0	.....	.....	.....	.....	149 miles double track. 276 " single "
Great Northern of.....	Ireland . .	503	70 000	64.0	36.0	.....	.....	.....	.....	136 " double " 367 " single "
Great Southern and Western of....	Ireland . .	478	80 000	60.0	40.0	.....	.....	.....	.....	195 " double " 283 " single "
All the Railroads, average of....	United States	110 414	62 176	59.7	40.3	0.0124	0.0242	.....	.....	59.7 was the average percentage of 1880, as per census. No reliable data for all the roads later.
Baltimore and Ohio, " Main Stem"	"	553	110 040	61.3	38.7	.....	.....	.....	.....	
Penn. Railroad—Penn. Division...	"	1 313	.....	63.5	36.5	0.00819	0.02448	.....	.....	
" Phil. and Erie " ...	"	288	.....	66.0	34.0	0.00624	0.03048	.....	.....	
" New Jersey " ...	"	435	.....	46.0	54.0	0.01471	0.02074	.....	.....	
" Av. of the three above Divisions.	"	2 036	.....	58.0	42.0	.....	.....	.....	.....	
Phila., Wilmington and Baltimore.	"	413	109 000	55.0	45.0	.....	.....	.....	.....	
Boston and Albany, 1884.....	"	384	101 840	58.0	42.0	0.00665	0.0231	.....	.....	Coal very high on this line.
Delaware, Lackawan'a and W., 1884	"	400	.....	50.0	50.0	0.00902	0.0224	.....	.....	
L. Shore and Michigan Soth'n, "	"	1 339	.....	51.0	49.0	0.0068	0.02189	.....	.....	
N. Y. Cen. and Hudson R., "	"	993	.....	52.0	48.0	0.0083	0.0194	.....	.....	
New York, L. Erie and Western, "	"	1 029	.....	50.0	50.0	0.00719	0.02168	.....	.....	

TABLE No. 18.

Exact copy of the part of the Semi-Annual Report of the London and North Western Railway relating to Traffic and Operating Expenses for the Six Months ending June 30th, 1883.

DR. REVENUE ACCOUNT—HALF-YEAR ENDING JUNE 30TH, 1883. CR.

Abstracts:	£ s. d	June 30th, 1882.
(A) To Maintenance of Way, Works and Stations	405 364 14 8	403 029
(B) To Locomotive Power	599 742 2 6	556 960
(C) To Carriage and Wagon Repairs	158 966 12 1	163 981
To Mileage of Carriages and Wagons of other Companies	13 823 2 4	12 291
(D) To Traffic Expenses	952 194 1 2	928 600
(E) To General Charges	114 212 17 5	109 460
To Law Charges	14 647 7 9	15 312
To Parliamentary Expenses	12 000 0 0	8 000
To Compensation for Accidents and Losses: Passengers £10 036 4 3; Goods, etc. 17 260 12 0	27 296 16 3	34 087
To Rates and Taxes	106 305 9 4	104 283
To Government Duty	64 953 4 10	63 115
To North Union Railway—Proportion of Joint Expense	6 995 16 2	5 132
To Birkenhead Railway—Proportion of Joint Expense	18 850 17 5	18 894
To Shrewsbury and Hereford Railway—Proportion of Joint Expense	9 736 15 1	12 392
To Shrewsbury and Wellington Railway—Proportion of Joint Expense	2 174 15 10	2 207
To Shrewsbury and Welshpool Railway—Proportion of Joint Expense	1 253 13 1	2 828
To Vale of Towy Railway—Proportion of Joint Expense	1 958 17 8	2 251
(F) To Steamboat Expenses	34 276 10 9	36 169
To Depreciation and Insurance of Steamboats	10 000 0 0	10 000
To Lancaster Canal Expenses	2 447 18 3	2 530
To Huddersfield Canal Expenses	2 569 18 4	3 108
	2 559 771 10 11	2 494 629
To Balance carried to Net Revenue Account	2 437 379 16 4	2 408 997
	£4 997 151 7 3	4 903 626

	Class.	No.	Amount. £ s. d.	£ s. d.	£ s. d.	June 30th, 1882.
By Traffic, viz.: Passengers	First	1 186 134	256 482 5 2			
	Second	1 962 244	173 908 13 7			
	Third	22 347 872	998 781 13 4			
	Season	19 578	72 703 17 9			
				1 501 876 9 10		
Parcels, Horses, Carriages and Dogs				259 936 10 6		
Mails				75 957 0 0		
					1 837 770 0 4	1 791 421
Merchandise			£1 966 366 6 10			
Less Expenses of Collection and Delivery of Goods by Agents			130 264 6 2			
				1 836 102 0 8		
Live Stock				79 465 8 6		
Minerals				1 168 120 2 8		
					3 083 687 11 10	3 039 680
					4 921 457 12 2	4 831 110
By Transfer Fees					724 2 6	742
						92 449
By Rents of Land and Buildings				97 112 7 3		20 675
Less Repairs and Sundries				22 142 14 8		
					74 969 12 7	71 774
					£4 997 151 7 3	4 903 626

TABLE No. 18.—*Continued.*

(A) MAINTENANCE OF WAY, ETC.			30th June, 1882.
	£ s. d.		
Salaries, Office Expenses and General Superintendence	£19 841 8 4		18 559
Maintenance and Renewal of Permanent Way, viz.:			
Wages	£138 979 0 7		130 290
Materials	48 715 7 10		55 337
Engines Ballasting	9 979 2 0		9 815
	197 673 10 5		195 442
Repairs of Approach Roads, Bridges, Signals, and other Works	57 515 16 7		57 699
Repairs of Stations and Buildings	35 229 4 3		31 991
		310 259 19 7	303 691
Rebuilding Stations		15 000 0 0	15 000
Repairs—Garston and Widnes Docks and Holyhead Harbor		6 017 3 3	3 953
" —St. Helens and Coalport Canals		2 825 19 7	2 899
Sundry Joint Lines and Stations		17 138 18 3	12 744
Signals, Sidings and Sundry Works		54 122 14 0	64 742
Miles Maintained : Double or more 1 310			
Single 374			
1 684			
		£405 364 14 8	403 029

(B) LOCOMOTIVE POWER.			30th June, 1882.
	£ s. d.		
Salaries, Office Expenses and General Superintendence		19 137 11 0	18 441
Running Expenses :			
Wages connected with the Working of Locomotive Engines	£227 274 6 3		214 795
Coal and Coke	135 437 19 4		123 419
Water	4 996 3 5		4 837
Oil, Tallow and other Stores	22 679 14 1		22 047
		390 388 3 1	365 098
Repairs and Renewals of Engines, Tools, etc.:			
Wages	£89 705 3 8		88 311
Materials	93 487 2 6		77 911
		183 192 6 2	166 222
Special Expenditure		11 009 18 5	15 839
Stationary Engines, Liverpool Tunnels		4 404 19 7	2 225
		608 132 18 3	567 825
Cr., Mileage of Sundry Trains	£12 085 9 4		
Less—Cromford and High Peak Working	3 694 13 7		
		8 390 15 9	10 865
		£599 742 2 6	556 990

(C) REPAIRS AND RENEWALS OF CARRIAGES AND WAGONS.			30th June, 1882.
	£. s. d.		
Carriages—Salaries, Office Expenses and General Superintendence	£1 636 18 10		1 624
Wages	37 429 11 4		33 480
Materials	53 197 7 9		58 819
		92 263 17 11	93 923
Wagons—Salaries, Office Expenses and General Superintendence	£2 382 6 3		2 382
Wages	17 872 9 1		18 047
Materials	46 447 18 10		49 629
		66,702 14 2	70,058
		£158 966 12 1	163 981

TABLE No. 18.—*Continued.*

STORES.				30th June, 1882.
	£	s.	d.	
Locomotive Department Stores	315 709	13	3	306 826
Locomotive Department Works in progress and Sundries	181 530	15	9	178 069
Carriage Department Stores	51 920	15	5	54 980
Carriage Department Sundries in process of manufacture	17 676	3	5	19 368
Wagon Department Stores	107 071	11	6	97 212
Wagon Department Sundries in process of manufacture	14 138	9	2	10 691
Permanent Way Stores	418 727	5	7	370 700
Engineering Stores	63 894	6	9	68 761
General Stores	27 224	9	11	29 387
Steamboat Stores	9 148	18	1	12 659
Telegraph Stores	14 261	9	5	10 793
	1 221 303	18	3	1 159 446
Engineering Stores, Joint Lines	36 300	14	8	31 595
Total	£1 257 604	12	11	1 191 041

(D) TRAFFIC EXPENSES.						30th June, 1882.
				£	s. d.	
Salaries, Wages, etc., Coaching and Police Departments	£224 209	9	6			215 044
Fuel, Lighting, Water and General Stores	59 000	13	7			60 670
Clothing	13 475	7	8			11 848
Printing, Stationery and Tickets	15 408	4	10			16 385
Joint Station Expenses	20 646	16	9			20 787
Horses, Harness, Parcel Carts, Provender, etc	11 836	9	1			11 389
				344 577	1 5	336 123
Salaries, Wages, etc., Merchandise Department	£419 710	8	10			411 042
Fuel, Lighting, Water, Grease and General Stores	28 480	6	5			26 017
Clothing	864	2	6			666
Printing, Stationery, etc	8 749	17	1			9 354
Horses, Harness, Vans, Provender, etc	96 802	7	7			99 322
Wagon Covers, Ropes, etc	12 495	14	9			11 307
Joint Station Expenses	9 502	18	9			6 746
Agents' Commission	1 895	10	6			1 990
Hoists, Hydraulic Cranes, etc	29 115	13	4			26 033
				607 616	19 9	592 477
				£952 194	1 2	928 600

(E) GENERAL CHARGES.						30th June, 1882.
				£	s. d.	
Direction				4 537	10 0	4 538
Auditors and Public Accountants				900	17 2	993
Salaries of Secretary, General Manager, Accountants and Clerks	£32 400	8	10			
Office Expenses Sundries	7 318	4	0			
				39 718	12 10	39 301
Advertising				2 039	17 6	1 670
Fire Insurance				6 905	6 8	6 905
Electric Telegraph Expenses				18 457	0 6	17 256
Railway Clearing House Expenses				15 289	14 10	14 446
Contribution to Superannuation Fund				6 294	13 2	6 279
Schools				1 162	6 8	943
Traveling Expenses, Directors, Managers, etc				1 688	16 5	1 869
Provident and Insurance Societies and Pension Fund				9 252	19 8	8 047
Sundry Expenses, not classed				7 965	2 0	7 213
				£114 212	17 5	109 460

TABLE No. 18.—*Continued.*

(F) STEAMBOAT EXPENSES.	£ s. d.	30th June, 1882.
Salaries of Commanders, Wages of Crews, etc.	19 142 5 11	19 310
Repairs, Ship Stores, etc.	6 118 14 1	5 798
Coal and Engineers' Stores	10 014 11 5	10 937
Light and Harbor Dues	7 089 9 6	7 176
General Charges	1 959 9 10	2 804
	44 324 10 9	46 025
Less—Cost of Working Greenore Service	10 048 0 0	9 856
	£34 276 10 9	36 169

STEAMBOAT DEPRECIATION AND INSURANCE ACCOUNT.

	£ s. d.		£ s. d.
To Balance 31st Dec. last	1 180 10 10	By Amount set aside from Revenue this half-year	10 000 0 0
To Expended this half-year	176 13 7		
To Balance	8 642 15 7		
	£10 000 0 0		10 000 0 0

MILEAGE STATEMENT.

	Miles Authorized.	Miles Constructed.	Miles Constructing or to be Constructed.	Miles Worked by Engines.	30th June, 1882.
Lines owned by Company	1 546¼	1 505	41½	1 499¼	1 490
Lines partly owned	74½	73½	1	140⅝	140⅝
Lines Leased or Rented	146½	142⅜	3⅜	213⅜	213¾
Total	1 767½	1 721	46¼	1 854	1 844½
Lines Worked	54	54	....	61⅞	51¼
Foreign Lines Worked over	....	....	....	560¼	569½
Total	1 821¼	1 775	46¼	2 476½	2 465½

STATEMENT OF TRAIN MILEAGE.

Passenger Trains	8 969 727	8 598 244
Goods and Mineral Trains	9 528 331	9 245 421
Total	18 498 058	17 843 665

TABLE No. 19.—*Continued.*

(A) MAINTENANCE OF WAY, ETC.			31st Dec., 1882.
Salaries, Office Expenses and General Superintendence	£19 668 13 9	£ s. d.	18 622
Maintenance and Renewal of Permanent Way, viz.:			
Wages	£138 169 17 2		132 483
Materials	133 968 14 8		135 557
Engines Ballasting	11 771 15 1		10 989
	283 910 6 11		279 029
Repairs of Approach Roads, Bridges, Signals and other Works	62 614 10 7		66 057
Repairs of Stations and Buildings	36 167 8 10		41 651
		402 361 0 1	405 359
Rebuilding Stations		20 000 0 0	20 000
Repairs—Garston and Widnes Docks and Holyhead Harbor		7 959 11 9	5 360
Repairs—St. Helens and Coalport Canals		3 326 9 1	2 638
Sundry Joint Lines and Stations		14 619 3 2	14 668
Signals, Sidings and Sundry Works		72 972 16 2	70 269
Miles Maintained:			
Double or more	1 313		
Single	374		
	1 687	£521 239 0 3	518 294

(B) LOCOMOTIVE POWER.			31st Dec., 1882.
		£ s. d.	
Salaries, Office Expenses and General Superintendence		18 961 10 7	19 204
Running Expenses:			
Wages connected with the Working of Locomotive Engines	£231 897 17 7		222 446
Coal and Coke	141 819 12 7		129 778
Water	4 728 2 8		4 563
Oil, Tallow and other Stores	27 292 15 2		22 725
		405 738 8 0	379 512
Repairs and Renewals of Engines, Tools, etc.:			
Wages	£90 774 9 6		94 064
Materials	93 885 4 8		84 624
		184 659 14 2	178 688
Special Expenditure		15 471 12 10	27 022
Stationary Engines, Liverpool Tunnels		3 653 3 2	2 167
Cr., Mileage of Sundry Trains	£14 288 6 10	628 484 8 9	606 593
Less—Cromford and High Peak Working	3 313 2 4		
		10 975 4 6	11 631
		£617 509 4 3	594 962

(C) REPAIRS AND RENEWALS OF CARRIAGES AND WAGONS.			31st Dec., 1882.
		£ s. d.	
Carriages—Salaries, Office Expenses and General Superintendence	£1 675 1 10		1 601
Wages	34 121 6 5		34 680
Materials	62 566 19 3		60 532
		98 363 7 6	96 813
Wagons—Salaries, Office Expenses and General Superintendence	£2 328 6 10		2 328
Wages	18 518 14 10		20 486
Materials	52 142 10 2		50 894
		72 989 11 10	73 708
		£171 352 19 4	170 521

TABLE No. 19.

Exact copy of the part of the Semi-Annual Report of the London and North Western Railway relating to Traffic and Operating Expenses for the six months ending December 31st, 1883.

DR. REVENUE ACCOUNT—HALF-YEAR ENDING DECEMBER 31ST, 1883. CR.

Abstracts:	£ s. d.	Dec. 31st, 1882.		Class.	No.	Amount.	£ s. d.	£ s. d.	Dec. 31st, 1882.
(A) To Maintenance of Way, Works and Stations	521 239 0 3	518 294	By Traffic, viz.: Passengers..	First.	1 246 005	303 003 2 9			
(B) To Locomotive Power	617 509 4 3	594 962		Second...	2 093 609	220 687 9 3			
(C) To Carriage and Wagon Repairs	171 352 19 4	170 522		Third....	25 335 955	1 233 774 15 4			
To Mileage of Carriages and Wagons of other Companies	11 431 5 3	11 361		Season...	17 260	57 969 5 0	1 815 434 12 4		
(D) To Traffic Expenses	964 726 9 2	969 175	Parcels, Horses, Carriages and Dogs				292 072 10 8		
(E) To General Charges	127 374 7 1	119 203	Mails				78 540 15 10		
To Law Charges	13 490 4 5	12 499						2 186 047 18 10	2 111 694
To Parliamentary Expenses	12 000 0 0	6 000							
To Compensation for Accidents and Losses: Passengers....£19 397 10 0 Goods, etc..... 20 922 17 8	40 320 7 8	34 225	Merchandise			£1 998 720 14 11			
			Less Expenses of Collection and Delivery of Goods by Agents			124 962 10 1			
To Rates and Taxes	121 862 5 6	108 868							
To Government Duty	59 865 0 4	77 416					1 873 758 4 10		
To North Union Railway—Proportion of Joint Expense	7 738 1 3	7 035							
To Birkenhead Railway—Proportion of Joint Expense	18 834 0 0	16 997							
To Shrewsbury and Hereford Railway—Proportion of Joint Expense	10 666 8 10	11 927	Live Stock				122 473 10 7		
To Shrewsbury and Wellington Railway—Proportion of Joint Expense	2 553 8 0	1 864	Minerals				1 189 076 16 2	3 185 308 11 7	3 164 674
To Shrewsbury and Welshpool Railway—Proportion of Joint Expense	1 344 18 6	3 351						5 371 356 10 5	5 276 368
To Vale of Towy Railway—Proportion of Joint Expense	2 103 6 6	2 212							
(F) To Steamboat Expenses	37 188 12 5	35 129							
To Depreciation and Insurance of Steamboats	15 000 0 0	15 000							
To Lancaster Canal Expenses	2 267 7 11	2 492	By Transfer Fees					638 3 5	701
To Huddersfield Canal Expenses	3 715 5 10	2 809							
	2 762 582 12 6	2 721 341	By Rents of Land and Buildings				108 829 19 8		108 013
			Less Repairs and Sundries				24 171 13 1		22 920
To Balance carried to Net Revenue Account	2 694 070 7 11	2 640 821						84 658 6 7	85 093
	£5 456 653 0 5	5 362 162						£5 456 653 0 5	5 362 162

TABLE No. 19.—*Continued.*

STORES.			31st Dec., 1882.
		£ s. d.	
Locomotive Department Stores		322 450 18 6	321 355
Do. Work in progress, and Sundries		191 302 9 0	198 768
Carriage Department Stores		54 448 13 6	56 251
Do. Sundries in process of manufacture		41 875 18 6	18 788
Wagon Department Stores		112 883 10 5	97 500
Do. Sundries in process of manufacture		16 143 7 5	14 601
Permanent Way Stores		441 260 2 4	426 865
Engineering Stores		35 526 2 3	38 544
General Stores		28 305 14 7	24 905
Steamboat Stores		10 302 0 9	9 280
Telegraph Stores		14 978 9 5	16 086
		1 269 477 6 8	1 222 943
Engineering Stores Joint Lines		36 455 15 4	29 171
Total		£1 305 933 2 0	1 252 114

(D) TRAFFIC EXPENSES.			31st Dec. 1882.
		£ s. d.	
Salaries, Wages, &c., Coaching and Police Departments	£236 307 18 7		230 377
Fuel, Lighting, Water and General Stores	57 194 10 2		59 032
Clothing	9 915 5 9		8 946
Printing, Stationery and Tickets	19 597 12 1		18 726
Joint Station Expenses	18 987 15 4		21 373
Horses, Harness, Parcel Carts, Provender, &c	10 983 17 10		14 087
		352 986 19 9	352 541
Salaries, Wages, &c., Merchandise Department	427 376 0 3		423 866
Fuel, Lighting, Water, Grease and General Stores	29 630 5 11		27 886
Clothing	712 2 4		841
Printing, Stationery, &c	9 198 15 5		8 8 2
Horses, Harness, Vans, Provender, &c	91 105 6 7		99 024
Wagon Covers, Ropes, &c	10 487 18 5		13 425
Joint Station Expenses	10 988 6 11		11 834
Agents' Commission	1 890 9 9		2 627
Hoists, Hydraulic Cranes, &c	30 350 3 10		28 289
		611 739 9 5	616 634
		£964 726 9 2	969 175

(E) GENERAL CHARGES.			31st Dec., 1882.
		£ s. d.	
Direction		4 537 10 0	4 538
Auditors and Public Accountants		897 12 1	964
Salaries of Secretary, General Manager, Accountants and Clerks	£32 522 10 2		
Office Expenses, Sundries	9 224 17 11		
		41 747 8 1	40 234
Advertising		3 629 17 3	3 029
Fire Insurance		13 810 13 4	13 811
Electric Telegraph Expenses		18 964 11 9	17 242
Railway Clearing House Expenses		16 909 15 6	15 911
Contribution to Superannuation Fund		6 312 1 1	6 113
Schools		1 303 4 6	872
Traveling Expenses, Directors, Managers, &c		1 544 8 5	1 792
Provident and Insurance Societies and Pension Funds		10 391 17 2	7 556
Sundry Expenses, not classed		7 325 7 11	7 141
		£127 374 7 1	119 203

TABLE No. 19.—*Continued.*

(F.) STEAMBOAT EXPENSES.		31st Dec., 1882.
	£ s. d.	
Salaries of Commanders, Wages of Crews, &c	20 188 13 11	17 980
Repairs, Ship Stores, &c	6 888 3 3	5 292
Coal and Engineers' Stores	10 156 3 11	11 247
Light and Harbor Dues	7 289 10 4	8 260
General Charges	2 651 13 0	2 398
	47 174 4 5	45 177
Less—Cost of Working Greenore Service	9 985 12 0	10 048
	£37 188 12 5	35 129

STEAMBOAT DEPRECIATION AND INSURANCE ACCOUNT.

	£ s. d.		£ s. d.
To Expended this Half-year	610 0 0	By Balance 30th June last	8 642 15 7
To cost of new Cattle Steamer Holyhead, lost 30th October last	35 817 3 11	By Amount set aside from Revenue this Half-year	15 000 0 0
		By Balance	12 784 8 4
	£36 427 3 11		£36 427 3 11

MILEAGE STATEMENT.

	Miles Authorized.	Miles Constructed.	Miles Constructing or to be Constructed.	Miles Worked by Engines.	31st Dec., 1882.
Lines owned by Company	1 570½	1 524¼	46¼	1 519	1 493
Lines partly owned	80½	79½	1	152	140¾
Lines Leased or Rented	146½	142¾	3¾	213¾	213¾
Total	1 797½	1 746½	51	1 884¾	1 847½
Lines Worked	30¾	30¾	..	33¾	51½
Foreign Lines Worked over	..	..	..	562¾	560¾
Total	1 828¼	1 777¼	51	2 481¼	2 459¾

STATEMENT OF TRAIN MILEAGE.

Passenger Trains	9 796 002	9 389 561
Goods and Mineral Trains	9 802 718	9 547 258
Total	19 598 720	18 936 819

TABLE No. 20.

Exact copy of the part of the Semi-Annual Report of the London and North Western Railway relating to Traffic and Operating Expenses for the six months ending June 30th, 1884.

DR. REVENUE ACCOUNT—HALF-YEAR ENDING JUNE 30TH, 1884. CR.

Abstracts:	£ s. d.	June 30th, 1883.
(A) To Maintenance of Way, Works and Stations	407 085 19 5	405 365
(B) To Locomotive Power	613 389 1 4	599 742
(C) To Carriage and Wagon Repairs	185 718 8 6	158 967
To Mileage of Carriages and Wagons of other companies	13 979 4 4	13 823
(D) To Traffic Expenses	941 888 6 10	952 194
(E) To General Charges	115 336 13 6	114 213
To Law Charges	13 789 8 5	14 647
To Parliamentary Expenses	11 500 0 0	12 000
To Compensation for Accidents and Losses: Passengers.. £11 504 0 8; Goods, etc.. 19 353 2 7	30 857 3 3	27 297
To Rates and Taxes	108 961 18 9	106 305
To Government Duty	23 752 10 4	64 953
To North Union Railway—Proportion of Joint Expenses	7 258 4 9	6 996
To Birkenhead Railway—Proportion of Joint Expenses	17 672 19 6	18 851
To Shrewsbury and Hereford Railway—Proportion of Joint Expenses	11 300 4 9	9 737
To Shrewsbury and Wellington Railway—Proportion of Joint Expenses	1 827 7 8	2 175
To Shrewsbury and Welshpool Railway—Proportion of Joint Expenses	2 263 13 1	1 254
To Vale of Towy Railway—Proportion of Joint Expenses	1 916 13 3	1 959
(F) To Steamboat Expenses	37 615 19 2	34 276
To Depreciation and Insurance of Steamboats	10 000 0 0	10 000
To Lancaster Canal Expenses	2 141 4 8	2 448
To Huddersfield Canal Expenses	2 908 14 9	2 569
	2 561 163 16 3	2 559 771
To Balance carried to Net Revenue Account	2 353 221 17 7	2 437 380
	£4 914 385 13 10	4 997 151

By Traffic, viz.:	Class.	No.	Amount.	£ s. d.	£ s. d.	£ s. d.	June 30th, 1883.
Passengers..	First.	1 139 500	241 533 15 7				
	Second.	1 946 000	172 710 13 2				
	Third.	22 841 751	1 007 949 19 8				
	Season.	20 119	71 947 6 0				
				1 494 141 4 5			
Parcels, Horses, Carriages and Dogs				282 549 14 2			
Mails				80 288 18 7		1 856 979 17 2	1 837 770
Merchandise			£1 922 229 17 0				
Less Expenses of Collection and Delivery of Goods by Agents..			118 691 15 10				
				1 803 538 1 2			
Live Stock				86 413 1 11			
Minerals				1 092 490 16 1		2 982 441 19 2	3 083 687
						4 839 421 16 4	4 921 457
By Transfer Fees						756 6 8	724
By Rents of Land and Buildings				98 143 17 0			97 112
Less Repairs and Sundries				23 936 6 2			22 142
						74 207 10 10	74 970
						£4 914 385 13 10	4 997 151

TABLE No. 20.—*Continued.*

(A) MAINTENANCE OF WAY, &c.			30th June, 1883.
Salaries, Office Expenses and General Superintendence	£20 018 9 8	£ s. d.	19 841
Maintenance and Renewal of Permanent Way, viz.:			
Wages	£136 112 18 2		138 979
Materials	48 402 7 1		48 716
Engines Ballasting	10 270 18 5		9 979
	194 786 3 8		197 674
Repairs of Approach Roads, Bridges, Signals and other Works	62 235 7 6		57 516
Repairs of Stations and Buildings	33 679 4 9		35 229
		310 719 5 7	310 260
Rebuilding Stations		15 000 0 0	15 000
Repairs—Garston and Widnes Docks and Holyhead Harbor		7 614 17 1	6 017
Do. St. Helens and Coalport Canals		3 380 16 8	2 826
Sundry Joint Lines and Stations		16 467 1 2	17 139
Signals, Sidings and Sundry Works		53 903 18 11	54 123
Miles Maintained: Double or more 1 321			
Single 370			
	1 691	£407 085 19 5	405 365

(B) LOCOMOTIVE POWER.			30th June, 1883.
		£ s. d.	
Salaries, Office Expenses and General Superintendence		19 985 6 8	19 138
Running Expenses:			
Wages connected with the Working of Locomotive Engines	£225 699 9 11		227 274
Coal and Coke	142 353 15 10		135 438
Water	4 237 5 3		4 996
Oil, Tallow and other Stores	24 071 15 0		22 680
		396 362 6 0	390 388
Repairs and Renewals of Engines, Tools, &c.:			
Wages	£88 079 3 2		89 705
Materials	105 272 5 3		93 487
		193 351 8 5	183 192
Special Expenditure		10 914 14 0	11 010
Stationary Engines, Liverpool Tunnels		3 537 13 6	4 405
		624 151 8 7	608 133
Cr., Mileage of Sundry Trains	£15 194 15 10		
Less—Cromford and High Peak Working	4 432 8 7		
		10 762 7 3	8 391
		£613 389 1 4	599 742

(C) REPAIRS AND RENEWALS OF CARRIAGES AND WAGONS.			30th June, 1883.
		£ s. d.	
Carriages—Salaries, Office Expenses and General Superintendence	£1 752 8 4		1 637
Wages	38 523 4 6		37 430
Materials	78 185 17 4		53 197
		118 461 10 2	92 264
Wagons—Salaries, Office Expenses and General Superintendence	£2 396 12 9		2 382
Wages	18 728 3 7		17 873
Materials	46 132 2 0		46 448
		67 256 18 4	66 703
		£185 718 8 6	158 967

TABLE No. 20.—*Continued.*

STORES.		30th June, 1883.
	£ s. d.	
Locomotive Department Stores	316 744 6 1	315 710
Do. Work in progress, and Sundries	186 327 4 7	181 531
Carriage Department Stores	58 834 17 7	51 921
Do. Sundries in process of manufacture	38 508 4 1	17 676
Wagon Department Stores	118 707 4 3	107 072
Do. Sundries in process of manufacture	20 649 16 6	14 138
Permanent Way Stores	458 299 13 6	418 727
Engineering Stores,	68 887 3 11	63 894
General Stores	29 131 12 9	27 224
Steamboat Stores	14 404 4 8	9 149
Telegraph Stores	14 492 0 5	14 261
	1 324 986 8 4	1 221 303
Engineering Stores, Joint Lines	33 539 16 6	36 301
Total	£1 358 526 4 10	1 257 604

(D) TRAFFIC EXPENSES.			30th June, 1883.
		£ s. d.	
Salaries, Wages, &c., Coaching and Police Departments	£230 779 16 7		224 209
Fuel, Lighting, Water and General Stores	62 956 11 1		59 001
Clothing	12 547 0 9		13 475
Printing, Stationery and Tickets	14 865 3 8		15 408
Joint Station Expenses	18 689 16 7		20 647
Horses, Harness, Parcel Carts, Provender, &c.	11 648 18 5		11 837
		351 487 7 1	344 577
Salaries, Wages, &c., Merchandise Department	416 285 8 7		419 710
Fuel, Lighting, Water, Grease and General Stores	28 853 19 5		28 480
Clothing	735 1 6		864
Printing, Stationery, &c.	9 024 0 4		8 750
Horses, Harness, Vans, Provender, &c.	82 721 0 4		96 802
Wagon Covers, Ropes, &c.	10 144 19 6		12 496
Joint Station Expenses	10 834 13 11		9 503
Agents' Commission	1 964 11 8		1 896
Hoists, Hydraulic Cranes, &c.	29 837 4 6		29 116
		590 400 19 9	607 617
		£941 888 6 10	952 194

(E) GENERAL CHARGES.			30th June, 1883.
		£ s. d.	
Direction		4 537 10 0	4 538
Auditors and Public Accountants		913 18 4	901
Salaries of Secretary, General Manager, Accountants and Clerks	£33 508 2 0		
Office Expenses, Sundries	7 455 9 0		
		40 963 11 0	39 718
Advertising		1 964 15 2	2 040
Fire Insurance		6 905 6 8	6 905
Electric Telegraph Expenses		18 176 2 10	18 457
Railway Clearing House Expenses		14 798 2 4	15 290
Contribution to Superannuation Fund		6 472 13 7	6 295
Schools		1 185 13 5	1 162
Traveling Expenses, Directors, Managers, &c.		1 655 9 9	1 689
Provident and Insurance Societies and Pension Funds		11 662 14 11	9 253
Sundry Expenses, not classed		6 100 15 6	7 965
		£115 336 13 6	114 213

TABLE No. 20.—*Continued.*

(F) STEAMBOAT EXPENSES.		30th June, 1883.
	£ s. d.	
Salaries of Commanders, Wages of Crews, &c	20 203 12 4	19 142
Repairs, Ship Stores, &c	7 002 13 3	6 119
Coal and Engineers' Stores	10 784 15 3	10 015
Light and Harbor Dues	7 181 11 2	7 089
General Charges	2 363 7 2	1 960
	47 535 19 2	44 325
Less—Cost of Working Greenore Service	9 920 0 0	10 048
	£37 615 19 2	34 277

STEAMBOAT DEPRECIATION AND INSURANCE ACCOUNT.

	£ s. d.		£ s. d.
To Balance 31st Dec. last	12 784 8 4	By Amount set aside from Revenue this Half-year	10 000 0 0
To Expended this Half-year	438 16 9	By Balance	3 223 5 1
	£13 223 5 1		£13 223 5 1

MILEAGE STATEMENT.

	Miles Authorized.	Miles Constructed.	Miles Constructing or to be Constructed.	Miles Worked by Engines.	30th June, 1883.
Lines owned by Company	1 570½	1 528¾	41¼	1 523½	1 499½
Do. partly owned	80½	79½	1	152	140¾
Do. Leased or Rented	146½	142¾	3¼	213¼	213¾
Total	1 797½	1 751	46½	1 889½	1 854
Lines Worked	30¾	30¼	....	33¾	61¾
Foreign Lines Worked over	....	....	....	562¾	560¾
Total	1 828¼	1 781¾	46½	2 485¼	2 476½

STATEMENT OF TRAIN MILEAGE.

Passenger Trains	9 318 457	8 969 727
Goods and Mineral Trains	8 933 380	9 528 331
Total	18 251 837	18 498 058

TABLE No. 21.—*Continued.*

(A) MAINTENANCE OF WAY, &c.		£ s. d.	31st Dec., 1883.
Salaries, Office Expenses, and General Superintendence	£19 145 19 2		19 669
Maintenance and Renewal of Permanent Way, viz.:			
Wages	£136 848 19 3		138 169
Materials	135 880 11 9		133 969
Engines Ballasting	11 010 7 2		11 772
	283 739 18 2		283 910
Repairs of Approach Roads, Bridges, Signals, and other Works	70 029 13 1		62 615
Repairs of Stations and Buildings	40 745 10 2		36 167
		413 661 0 7	402 381
Rebuilding Stations		20 000 0 0	20 000
Repairs—Garston and Widnes Docks and Holyhead Harbor		6 719 5 0	7 960
Do. St. Helens and Coalport Canals		3 758 12 7	3 306
Sundry Joint Lines and Stations		17 420 3 4	14 619
Signals, Sidings, and Sundry Works		46 658 8 5	72 973
Miles Maintained:			
Double or more	1 334		
Single	370		
	1 704	£508 217 9 11	521 239

(B) LOCOMOTIVE POWER.		£ s. d.	31st Dec., 1883.
Salaries, Office Expenses, and General Superintendence		19 421 7 6	18 962
Running Expenses:			
Wages connected with the Working of Locomotive Engines	£238 585 18 2		231 898
Coal and Coke	140 763 11 0		141 819
Water	5 581 11 8		4 728
Oil, Tallow, and other Stores	27 256 11 6		27 293
		412 187 12 4	405 738
Repairs and Renewals of Engines, Tools, &c.:			
Wages	£90 523 2 4		90 774
Materials	96 598 6 4		93 885
		187 121 8 8	184 659
Special Expenditure		15 479 5 1	15 472
Stationary Engines, Liverpool Tunnels		3 724 9 1	3 653
		637 934 2 8	628 484
Cr., Mileage of Sundry Trains	£16 947 12 10		
Less—Cromford and High Peak Working	2 914 16 5		
		14 032 16 5	10 925
		£623 901 6 3	617 509

(C) REPAIRS AND RENEWALS OF CARRIAGES AND WAGONS.		£ s. d.	31st Dec., 1883.
Carriages—Salaries, Office Expenses and General Superintendence	£1 695 2 1		1 675
Wages	38 096 3 3		34 121
Materials	72 543 19 2		62 567
		112 335 4 6	98 363
Wagons—Salaries, Office Expenses and General Superintendence	£2 353 8 0		2 328
Wages	22 743 10 2		18 519
Materials	46 621 4 8		52 143
		71 718 2 10	72 990
		£184 053 7 4	171 353

TABLE No. 21.

Exact copy of the part of the Semi-Annual Report of the London and North Western Railway relating to Traffic and Operating Expenses for the six months ending December 31st, 1884.

DR. REVENUE ACCOUNT—HALF-YEAR ENDING DECEMBER 31ST, 1884. CR.

Abstracts:	£ s. d.	Dec. 31st, 1883.
(A) To Maintenance of Way, Works and Stations	508 217 9 11	521 239
(B) To Locomotive Power	623 901 6 3	617 509
(C) To Carriage and Wagon Repairs	184 053 7 4	171 353
To Mileage of Carriages and Wagons of other Companies	10 774 2 7	11 431
(D) To Traffic Expenses	958 739 9 5	964 726
(E) To General Charges	126 791 18 10	127 374
To Law Charges	13 449 10 8	13 490
To Parliamentary Expenses	12 000 0 0	12 000
To Compensation for Accidents and Losses: Passengers. £14 527 12 0 Goods, etc... 22 386 2 4	36 913 14 4	40 321
To Rates and Taxes	128 003 2 11	121 862
To Government Duty	28 578 17 8	59 865
To North Union Railway—Proportion of Joint Expense	8 060 1 4	7 738
To Birkenhead Railway—Proportion of Joint Expense	17 213 2 1	18 834
To Shrewsbury and Hereford Railway—Proportion of Joint Expense	11 935 6 3	10 667
To Shrewsbury and Wellington Railway—Proportion of Joint Expense	3 602 13 10	2 554
To Shrewsbury and Welshpool Railway—Proportion of Joint Expense	2 107 0 0	1 345
To Vale of Towy Railway—Proportion of Joint Expense	2 320 6 3	2 103
(F) To Steamboat Expenses	37 897 10 6	37 189
To Depreciation and Insurance of Steamboats	15 000 0 0	15 000
To Lancaster Canal Expenses	2 056 0 1	2 268
To Huddersfield Canal Expenses	2 647 9 7	3 715
	2 734 262 9 10	2 762 583
To Balance carried to Net Revenue Account	2 677 152 19 11	2 694 070
	£5 411 415 9 9	5 456 653

	Class.	No.	Amount.	£ s. d.	£ s. d.	Dec. 31st, 1883.
By Traffic, viz.: Passengers	First	1 180 832	289 514 8 2			
	Second	2 039 590	214 170 4 5			
	Third	25 788 360	1 273 754 3 4			
	Season	18 959	60 566 1 11			
				1 838 004 17 10		
Parcels, Horses, Carriages and Dogs				303 251 2 3		
Mails				83 182 13 6		
					2 224 438 13 7	2 1~6 048
Merchandise			£1 955 869 6 9			
Less Expenses of Collection and Delivery of Goods by Agents			120 076 19 6			
				1 835 792 7 3		
Live Stock				127 267 9 9		
Minerals				1 130 339 10 0		
					3 093 399 7 0	3 185 309
					5 317 838 0 7	5 371 357
By Transfer Fees					752 14 0	638
By Rents of Land and Buildings				116 879 13 4		108 830
Less Repairs and Sundries				24 054 18 2		24 172
					92 824 15 2	84 658
					£5 411 415 9 9	5 456 653

TABLE No. 21.—*Continued.*

Stores.		31st Dec., 1883.
	£ s. d	
Locomotive Department Stores	344 406 18 6	322 451
Do. Work in progress and Sundries	208 191 0 4	191 302
Carriage Department Stores	61 125 14 0	54 449
Do. Sundries in process of manufacture	28 740 5 6	41 876
Wagon Department Stores	120 237 4 6	112 884
Do. Sundries in process of manufacture	13 370 1 8	16 143
Permanent Way Stores	483 480 3 4	441 260
Engineering Stores	35 179 4 1	35 526
General Stores	27 350 15 8	28 306
Steamboat Stores	12 333 14 4	10 302
Telegraph Stores	13 637 0 10	14 978
	1 348 052 2 9	1 269 477
Engineering Stores, Joint Lines	35 482 11 6	36 456
Total	£1 383 534 14 3	1 305 933

(D) Traffic Expenses.			31st Dec., 1883.
		£ s. d.	
Salaries, Wages, &c., Coaching and Police Departments	£239 839 12 5		236 308
Fuel, Lighting, Water and General Stores	59 793 13 6		57 195
Clothing	10 456 4 7		9 915
Printing—Stationery and Tickets	20 503 4 11		19 598
Joint Station Expenses	16 327 7 7		18 987
Horses, Harness, Parcel Carts, Provender, &c.	10 336 13 8		10 984
		357 256 16 8	352 987
Salaries, Wages, &c., Merchandise Department	427 934 11 11		427 376
Fuel, Lighting, Water, Grease, and General Stores	28 836 1 5		29 680
Clothing	816 13 1		712
Printing, Stationery, &c	8 886 11 7		9 199
Horses, Harness, Vans, Provender, &c	87 838 18 9		91 105
Wagon Covers, Ropes, &c	9 497 2 4		10 488
Joint Station Expenses	12 035 14 5		10 988
Agents' Commission	2 010 5 0		1 891
Hoists, Hydraulic Cranes, &c	23,626 14 3		30 350
		601 482 12 9	611 739
		£958 739 9 5	964 726

(E) General Charges.			31st Dec., 1883.
		£ s. d.	
Direction		4 537 10 0	4 537
Auditors and Public Accountants		871 13 1	898
Salaries of Secretary, General Manager, Accountants and Clerks	£33 755 8 2		
Office Expenses, Sundries	8 814 14 2		
		42 570 2 4	41 747
Advertising		3 780 13 7	3 630
Fire Insurance		13 810 13 4	13 811
Electric Telegraph Expenses		18 051 3 5	18 965
Railway Clearing House Expenses		15 547 10 3	16 910
Contribution to Superannuation Fund		6 502 18 6	6 312
Schools		1 311 6 11	1 303
Traveling Expenses, Directors, Managers, &c		1 546 13 3	1 544
Provident and Insurance Societies and Pension Funds		12 124 13 3	10 392
Sundry Expenses, not classed		6 137 0 11	7 325
		£126 791 18 10	127 374

TABLE No. 21.—*Continued.*

(F) STEAMBOAT EXPENSES.		31st Dec., 1883.
	£ s. d.	
Salaries of Commanders, Wages of Crews, &c.	20 015 2 2	20 189
Repairs, Ship Stores, &c.	6 862 12 8	6 888
Coal and Engineers' Stores	10 773 10 5	10 156
Light and Harbor Dues	7 698 1 11	7 289
General Charges	2 596 3 4	2 652
	47 945 10 6	47 174
Less—Cost of Working Greenore Service	10 048 0 0	9 985
	£37 897 10 6	37 189

STEAMBOAT DEPRECIATION AND INSURANCE ACCOUNT.

	£ s. d.		£ s. d.
To Balance 30th June last....	3 223 5 1	By Amount set aside from Revenue this Half-year.....	15,000 0 0
To Balance.....	13 301 14 11	By sale of the Telegraph.....	1 525 0 0
	£16 525 0 0		£16 525 0 0

MILEAGE STATEMENT.

	Miles Authorized.	Miles Constructed	Miles Constructing or to be Constructed.	Miles Worked by Engines.	31st Dec., 1883.
Lines owned by Company...	1 576¼	1 542	34¼	1 532¼	1 519
Do. partly owned.....	86	85	1	163	152
Do. Leased or Rented.....	141	137	4	203	213¾
Total.....	1 803¼	1 764	39¼	1 898¼	1 884¾
Lines Worked.....	30¾	30¾	.....	33¾	33¾
Foreign Lines Worked over.	.....	.....	.....	562¾	562¾
Total	1 834	1 794¾	39¼	2 494¾	2 481¼

STATEMENT OF TRAIN MILEAGE.

Passenger Trains.....	10 258 096	9 796 092
Goods and Mineral Trains.....	9 438 932	9 802 718
Total.....	19 697 028	19 598 720

The Baltimore and Ohio Railroad.

This railroad has been selected by the author as a sample of the extreme type of the American system of railroad construction. In this we have a trunk line doing a very large and profitable traffic, constructed in an engineering sense through an extremely difficult country, which obliged the adoption of very sharp curves and steep grades. (See Plate I.)

It is perfectly safe to say that the curves on this road could not be operated safely at any reasonable speed by rolling stock built on the English type, with long and rigid wheel-base; and to have built this road with easy curves, suitable for it, the cost would have been many times what it has; in fact the cost would have been so great, that, as a commercial undertaking, it would have been impracticable. The operating expenses of the Baltimore and Ohio Railroad show that engineers generally have over-estimated the expense of operating sharp curves. The experience of this road shows that with properly constructed rolling stock the expense is comparatively small, and it can be safely operated at high speed.

This road attains a maximum summit elevation of 2 620 feet above the sea, with very sharp curves, yet the total cost of all the coal used on the main stem and branches, aggregating 553 miles, only cost $339 239.73, in a total operating expense of $6 147 658.80, or 5.5 per cent. The annual cost of fuel for each mile of road is $613, which is 6 per cent. interest on $10 216. So if it had been possible to have located a road that would *have saved all the fuel*, the engineer would have been only justified in expending $10 216 per mile additional to accomplish it.

This road has a great many disadvantages to contend with, and no advantages except cheap fuel, which is probably about the same price as on the English roads.

The data about the Baltimore and Ohio Railroad from which these Tables have been constructed, have been taken from the Company's Annual Report for 1883, pages 30 and 44; the line operated embracing the main stem, including Winchester and Potomac, Winchester and Strasburg, Strasburg and Harrisonburg, Metropolitan Branch, Washington City and Point Lookout, and Somerset and Cambria branches, making a total of 553 miles.

By referring to Table 12, showing the annual operating expenses of the Baltimore and Ohio Railroad, the percentage of all the expenses that are affected by good or bad construction amounts to	61.3
The English roads average	51.8
Balance against Baltimore and Ohio, per cent	9.5
Annual operating expenses of Baltimore and Ohio as per Table 12	$6 147 666
Annual saving in the working expenses, if the road had been constructed equal to the average English roads, $6 147 666 × .095	584 028
Ditto per mile	1 056
This annual saving per mile, capitalized at six per cent.	17 600

Thus it appears that the engineer would have been justified in spending $17 600 only additional, in making the road equal to the English standard of perfection. You can judge how far this would have gone in enlarging the many curves from 600 feet radii to a minimum of half a mile radii, to say nothing of the grade, and the summit elevation.

Mr. James L. Randolph, M. Am. Soc. C. E., Consulting Engineer of the Baltimore and Ohio Railroad, gives the author the following short description of the character of the line:

First Division.

Length of line ..... 100 miles.

Baltimore (50′ A. M. T.) to Martinsburgh (415′ A. M. T.)

The road has no notes of the line between Baltimore and Harper's Ferry. The seven miles next to Baltimore, and the forty-three next to Martinsburgh, have curves and tangents about the same as the Second Division. The other fifty miles, in the valley of the Patapsco, has strong curves and few tangents, and is similar to the Third Division.

Second Division.

Length of line	100 miles.
Martinsburgh (415′ A. M. T.) to Piedmont (919′ A. M. T.)	
Minimum radius of curves	1 000 feet.
Degrees turned	10 086 = 28 miles.
Total ascent westward	778 feet.
Total descent "	274 "
Maximum grade	39.6 per mile.
Running time, including stoppages, 2 hours 35 minutes, =	38.7 miles per hour.

Third Division.

Length of line..... 73¾ miles.
Piedmont (919′ A. M. T.) to Grafton (987′ A. M. T.)
Minimum radius of curves..... 600 feet.
33¾ miles curved, and 5 miles of this less than 650 feet radius.
Degrees turned..... 9 078.
Maximum grade on tangents..... 120 feet.
Maximum grade on curves of 650 feet radius..... 114 feet per mile.
Total ascent westward..... 2 402 "
Total descent " ..... 2 334 "
Running time, including stoppages, 2 hours 38 minutes, = ... 29.3 miles per hour.

Fourth Division.

Length of line..... 99¼ miles.
Grafton (987′ A. M. T.) to Wheeling (645′ A. M. T.)
Minimum radius of curves..... 600 feet.
Degrees turned..... 14 976.
Length of curved line..... 59¼ miles.
" " straight " ..... 40 "
Length of curvature less than 650 feet radius..... 5½ "
Maximum grade per mile..... 80 feet.
Total ascent westward..... 779 "
Total descent " ..... 1 121 "
Running time, including stoppages, 3 hours 14 minutes, = ... 30.8 miles per hour.
Elevation of summit above mean tide... 2 620 feet.

The Pennsylvania Railroad.

The officers of the Pennsylvania Railroad have kindly sent the author the data given below and that on Plate II. This railroad has justly become celebrated for its thorough management, cheap charges, both for freight and passengers, and the great attention paid to promoting the comfort of the latter.

As will be seen by the profile on Plate II, the road commences at tide level at Philadelphia, runs over an undulating country, crossing

the Blue Ridge at Gallitzin at an elevation of 2 154 feet above tide, terminating at Pittsburgh at 738 feet above tide.

PHILADELPHIA, August 11th, 1885.

MR. EDWARD BATES DORSEY,
Consulting Engineer,
The American Exhibition,
7 Poultry, London, E. C., Eng.

MY DEAR SIR,—I am pleased to acknowledge receipt of your favor of August 1st, with inclosure. I am glad to know you have taken so warm an interest in the comparison of the various railways throughout the world. There is a great deal that may be said in favor of our American railways, but I think a great deal of criticism can be made against them.

I note what you say, especially in reference to the difference in the compensation which the railway companies receive from different classes of passenger travel. This is a question to which I have given some thought. I am well satisfied that in America what we call parlor or sleeping-car traffic does not pay anything like the returns which the railway companies receive from ordinary passenger travel.

Very truly yours,

G. B. ROBERTS,
President.

PHILADELPHIA, June 11th, 1885.

EDWARD BATES DORSEY, Esq.,
Member of American Soc. C. E.,
London, Eng.

DEAR SIR,—Answering your letter of the 23d ult., which only reached me on the 8th inst., I send you herewith a blue print showing the grade line of the Pennsylvania Railroad from Pittsburgh to Philadelphia.

We have no plan of the line of the road on a small scale from which we could get you a print.

Our curves are generally light, though we have three or four on the line as high as eight degrees, and quite a number of six degrees, but the majority are less than four degrees.

I am not able at the present time to give you the percentage of curvature, but it certainly does not vary much from 33 per cent.

We also inclose you a lithograph of the cross-section of our road-bed, which, I think, has nearly as much effect on the economical working of a railroad as the curvature. We have learned by experience that the cheapest way to operate a railroad is to keep the track in first-class condition at all times.

Respectfully,

WM. H. BROWN,
Chief Engineer.

Cost of Construction Compared.

From the preceding tables, it appears that the average cost of railways in

England, in 1883 was, per mile.....	$202 227
In the United States "	62 176*
Excess in cost of the English.....	$140 051

In England, the land or right of way, Parliamentary expenses, block system, and crossings, cost much more than in the United States, but this is more than balanced by the saving in the price of labor, iron and steel, which constitute most of the cost of railroad construction. These items probably averaged during the construction of the roads one-third cheaper in England than in the United States.

Owing to the high prices of land, high charges for right of way, Parliamentary expenses, the almost universal use of the block system of signals, and the bridges at road-crossings, the cost of construction of the English railroads must necessarily be greater than the American.

Referring to Table 6, it appears that the percentage of the total annual operating expenses that is affected by good or bad construction is 7.9 per cent. greater on the average of all the American railroads than on the average of all the English roads—say eight per cent.

Table 16 shows that the total operating expenses for all the railways of England in 1883 was, per mile, $10 000, and in the United States, for all the railroads it was, per mile, $4 410.

If all the railroads of the United States had been as well constructed as in England, that is, at an additional cost of $140 051 per mile, there would have been saved eight per cent. of the annual operating expenses, which averaged $4 410 per mile; eight per cent. of this amounts to $353, which would have been the amount saved by expending $140 051 more per mile; this equals 0.25 per cent. on the additional cost. In other words, it would have required an additional expenditure in construction of $140 051 per mile, to save annually $353 in working expenses.

On the other hand, if the English railroads had been built on the American plan, at $140 051 per mile less cost, their working expenses would have been increased eight per cent., or $800 per mile annually. To save this, $140 051 has been expended per mile.

* Probably over one-third of this cost is made by inflated or watered stock. If all our railroads had been built as the English built theirs, where all the stock and bonds are sold at or near par, and nearly all the proceeds applied fairly and squarely to the construction or improvement of the roads, the average cost would not be over $40 000 per mile.

Poor's Manual for 1883, which is the best authority on the cost and working of the American railroads, at page iv says: "The actual cost of all the railways in the United States does not exceed $3 787 410 728; this divided by 121 592 miles = $31 148 average cost per mile."

OPERATING EXPENSES OF ENGLISH AND AMERICAN RAILROADS COMPARED.

Explanations.

In order to compare the cost of operating the two systems of railroads, and to ascertain if the greater cost of constructing the English roads gave a corresponding saving in working expenses, Tables Nos. 23, 24, 25 and 26 were computed. The data for the English roads were taken from their semi-annual reports and from the reports of the Board of Trade. Those for the American roads were taken from the reports of the Railroad Commissioners of the States of New York and Massachusetts, and in some cases from the reports of the companies, and in all cases can be considered reliable. In some cases it has been necessary to exercise discretion in analysing the accounts, as different companies have different forms for keeping their books. The railroads of the United Kingdom selected for comparison embrace all the leading roads. The American railroads selected are all located in a country naturally as rough and offering as great, if not greater, physical obstacles to cheap working than that in which the English roads are located. All roads or branches located in level or prairie country have been excluded.

The American roads selected have certainly no advantage in the physical character of the country they pass through.

In order to make the comparison more complete, the selected American roads embrace all kinds of roads, from the large trunk lines down to the small provincial or local roads. All, however, show very uniform workings.

Ton and Passenger Mileage Per Mile of Single Track.

Many prominent railroad men in England have told the author that it is necessary to run the freight trains at great speed in order to clear the track of their enormous business. In order to compare the traffic on the different roads, and to ascertain the force of this assertion, Table No. 24 was constructed, showing the analyses of the traffic of the principal railroads of each country. In this table all the tracks have been reduced to single track, and the ton and passenger mileage calculated upon this basis.

In all returns made by the English railroads, no mention is made of

third or fourth tracks, they being included under the general head of "double or more tracks."*

All the large companies of England have many miles of third and fourth tracks. For the want of reliable data these have not been calculated in their mileage; if these were added, their single track mileage would be largely increased, and, consequently, the traffic per mile would be proportionally reduced.

On the American railroads all tracks have been estimated and included in the single track.

In both countries all sidings and turnouts have been excluded.

The following recapitulation of Table No. 24 embraces the railroads having the most traffic.

TABLE No. 22.

Name of Railroad.	Mileage per Mile of Single Track.	
England.	Passenger.	Ton.
North Eastern	137 000	455 138
Midland	181 272	512 714
London and North Western	233 777	473 912
London, Chatham and Dover	634 036	195 906
London, Brighton and South Coast	547 328	192 778
Great Western	190 111	308 703
Average	320 587	356 525
United States.		
Boston and Albany	272 642	609 686
Boston and Providence	492 863	194 772
New York, New Haven and Hartford	516 694	314 359
New York Central and Hudson River	182 680	927 973
New York, Lake Erie and Western	114 982	1 216 913
Pennsylvania—Pennsylvania Railroad Division	130 235	1 594 898
Average	285 016	809 767

* The returns of the Board of Trade for 1883, in their Table No. 2A, gives the third and fourth track on some of the English railroads. Apparently in some cases sidings are included in this list. In order to be on the safe side, these additions have not been included in the estimate of single-track mileage; by including them, the single track stated in this paper would be increased as follows, viz.:

London and North Western	256 miles.	Great Northern	82 miles.
London and South Western	20 "	London, Brighton and South Coast	28 "
Midland	162 "	Great Western	38 "

From this it appears that the above six American railroads average more than double freight, with only eleven per cent. less passenger traffic than the six English.

The Pennsylvania ton mileage is more than three times that of the Midland, the highest ton mileage on the English list, while their passenger mileage is only 29 per cent. less.

The Boston, New Haven and Hartford in passenger traffic is 19 per cent. less than the London, Chatham and Dover, the highest on the English list, while their freight is 60 per cent. more.

The Boston and Albany has a larger passenger and freight traffic than any of the large or trunk lines of England.

From this showing, it does not appear necessary to run the trains on the English roads any faster in order to clear the tracks of traffic than on the American railroads.

Cost of Fuel.

The management of the railroads of the United Kingdom kindly furnished the author with the cost of coal used on their respective lines. For this information and other courtesies received from them, the author takes this opportunity to return his thanks. As this information was furnished confidentially, he does not feel justified in giving the exact figures. As a compromise, he has assumed the cost of coal on the lines north and west of London running into the coal-fields at 6s. 8d., or $1.60, per ton of 2 240 lbs., and on most of the roads south of London at 12s. 6d., or $3. In most cases this is somewhat higher than the actual prices given, in some few instances slightly lower, but in all cases sufficiently accurate for comparison. The price given is the cost of coal delivered to the roads, and does not include any charge for transportation on the road consuming it.

The cost of coal on the American railroads running into coal-fields has been taken as the same as on the English railroads running into coal-fields. *i. e.*, $1.60 per ton of 2 240 lbs.

The cost of coal consumed on the New York Central and Hudson River Railroad could not be ascertained from any official source. All through this paper it has been estimated as costing $3.20 per ton of 2 240 lbs.

Wages of Engine-Drivers and Firemen.

After careful inquiry, the average daily wages of the English engine-driver has been taken at 7s. = $1.68, and the American at $3.83. And

that of the English fireman at 4s. = $0.96, and the American at $1.99. To equalize these, the American wages should be reduced 54.6 per cent., but in order to be safe, they have only been reduced 50 per cent.

Repairs and Renewals of Locomotives.

The result as shown by this column in Table No. 23 was a great surprise to the author. He had the impression that the English locomotive on their smooth roads would require less repairs than the apparently weaker American locomotive on our rougher roads, some of which are far from being equal to the English standard.

The average cost of repairs and renewals of locomotives on fourteen railroads in the United Kingdom was 7.8 per cent. of the total operating expenses, against 5.7 per cent., the average of eight American railroads, or say, one-third less.

This is very strong and emphatic contradiction to the often repeated assertion, that the outside-cylinder engines will shake and twist themselves to pieces. A few days since, a prominent railroad man in England told the author that he had thoroughly tried the outside-cylinder engines, but had given them up on account of the great expense of repairs, "that they wriggled themselves to pieces."

As labor constitutes about one-half the cost of locomotive repairs, the wages paid in the United States should be reduced 50 per cent. to equalize them with those paid in England; the average percentage of the cost of repairs and renewals on the American railroads would then be 4.3 per cent. of the total operating expenses, against 7.8 per cent. on the railroads of the United Kingdom, or nearly one-half.

Maintenance of Way.

The average cost on fourteen English railways was 17.8 per cent. of the total operating expenses, while the average on eight American roads was 21.6 per cent., say 4 per cent. more. This apparently should be much larger when we consider the great amount of perishable materials that enter into our construction, the higher wages to our workmen, and our severe winter weather.

Passenger Cars.—Repairs and Renewals.

The cost of repairs and renewals of passenger cars is about the same in the two countries, the average on the English roads for 1 000 pas-

senger miles being $1.27, and on the American $1.34. As 86 per cent. of the travel in the United Kingdom is third class—on the carriages for this no attempt at ornament, finish or comfort is ever made, they being generally little better than our box cars, with cheap seats and cushions—the first cost and repairs should be very small, much less than on ours. This will make the repairs and renewals of the first-class carriages very large, probably over three times what ours cost. This is another argument in favor of the author's previous assertion, that first-class travel, as now conducted on the English railroads, does not pay.

Freight Cars.—Repairs and Renewals.

But little information can be derived from this comparison, owing to no distinction being made in the reports of the tonnage transported in the companies' cars, or in the cars of firms, or cars of outside companies. This makes the cost of repairs and renewals appear very small on trunk lines, and greater on small lines that run their own cars. As over 90 per cent. of the English freight cars are either coal or open flat cars, their cost of repairs and renewals may be less than ours.

Cost of General Management.

This is about the same in both countries, varying from one to two per cent.—averaging two per cent. of the total operating expenses.

Cost of Motive Power.

The author was very much astonished at the motive power costing nearly double on the English railroads what it does on the American, as shown by Table 23. He expected it would be much greater on the latter, owing to their inferior construction and alignment. In fact this comparison, showing the cost of motive power, was commenced to show the American engineer the economy in operating railroads with good road beds, etc., such as the English have, over the generally inferior constructed roads of the United States. To ascertain if this was not a singular coincidence in one year's business, Table No. 26 was constructed, which shows on five of the principal railroads of each country, for the years 1882, 1883 and 1884, the average annual total operating expenses, cost of fuel, cost of labor on motive power, and the cost of repairs and renewals of locomotives.

TABLE No. 23.

Analysis of the Operating Expenses of the Principal Railroads in the United Kingdom and United States.

NAME OF RAILROAD.	MOTIVE POWER — Fuel: Total Cost of Fuel, Including Expenses in connection with Locomotive Department. Items included.	Fuel: Cost: Total	Fuel: Cost: Per Ton of 2,240 lbs.	Fuel: Percentage on Total Operating Expenses: Amount Actually Paid.	Fuel: Percentage on Total Operating Expenses: on assumed basis that all Fuel Cost at rate of $1 60 per 2,240 lbs. of Coal	Wages of Engine-drivers and Firemen: Total Amount Actually Paid	Per Centages on Total Operating Expenses: Amount Actually Paid.	Per Centages on Total Operating Expenses: All American Wages reduced to Equivalent of £1 per Day, Equalize them with the English.	Per Centage of Cost of Fuel and Wages of Engine-drivers and Firemen to Total Operating Expenses: Amount Actually Paid	Per Centage of Cost of Fuel and Wages of Engine-drivers and Firemen to Total Operating Expenses: Reducing all Labor to £1 per Day of 2,240 lbs., and American Wages 50 Per Cent.	Repairs and Renewals of Locomotives: Total Amount Actually Paid	Repairs and Renewals of Locomotives: Percentage on Total Operating Expenses: Amount Actually Paid	Repairs and Renewals of Locomotives: Percentage on Total Operating Expenses: One half of the cost being Labor, which is reduced 50 per cent to Equalize it with the English Price	Percentage on Total Operating Expenses of Cost of Fuel, Engine-drivers and Firemen, and of Repairs and Renewals of Locomotives: Amount Actually Paid	Percentage on Total Operating Expenses of Cost of Fuel, Engine-drivers and Firemen, and of Repairs and Renewals of Locomotives: All Coal reduced to $1 60 per Ton, of 2,240 lbs., and all American Wages 50 per cent.	Maintenance of Way, Exclusive of Taxes. — Note.—This column, from reports of the English Companies, differs somewhat from Board of Trade Returns: Amount paid	Maintenance of Way, Exclusive of Taxes. — Percentage on Total Operating Expenses	Repairs and Renewals of Passenger Cars: Amount Paid	Repairs and Renewals of Freight Cars. — Total cost in Repairs and Renewals of Cars in Transporting 1,000 Passengers 1 Mile.	Repairs and Renewals of Freight Cars. — Total cost in Repairs and Renewals of Cars in Transporting 1,000 Tons 1 Mile.	Expenses of General Management, Directors, Clerks, Auditor, &c.: Amount Paid	Expenses of General Management, Directors, Clerks, Auditor, &c.: Percentage on Total Operating Expenses.	Taxes and Government Duties: Amount Paid	Taxes and Government Duties: Percentage on Total Operating Expenses.	Percentage of Total Operating Expenses to Gross Earnings, Taxes Included	Dividends Paid Per Cent	NAME OF RAILROAD.
UNITED KINGDOM, 1885																											UNITED KINGDOM, 1883.
Great Northern	$10,047,410	$696 975	$1 60	7 0	7 0	$903 670	9 0	9 0	16 2	16 0	$1,196,635	11 8	6 8	22 8	22 8	$1,122,840	11 2	$102,085	$1 40	$440,040	$ 88	$191,745	1 9	$502,275	6 0	56	4	Great Northern
North Eastern	17,250,150	1,150,780	1 60	6 7	6 7	1,640,675	9 5	9 5	16 2	16 2	1,994,480	11 6	11 6	27 8	27 8	3,136,040	19 3	509,750	1 65	365,388	92	262,400	1 5	1,091,490	5 8	71	8	North Eastern.
Midland	19,431,985	1,147,405	1 60	6 1	6 0	2,153,220	11 0	11 0	17 0	17 0	1,800,705	9 3	9 3	26 3	26 3	3,286,195	17 4	802,445	1 99	1,170,450	97	261,875	1 4	1,822,285	6 0	51	7	Midland.
London and North Western	26,140,425	1,567,267	1 60	5 4	5 3	2,296,860	8 8	8 8	14 1	14 1	1,809,240	7 0	7 0	21 1	21 1	4,163,020	15 7	1,345,175	1 28	678,000	40	578,995	1 5	1,867,841	6 0	51	7	London and North Western.
Great Western	18,578,720	1,147,880	1 60	5 4	5 4	1,584,205	8 4	8 4	13 8	13 8	1,365,100	9 8	9 8	23 1	23 6	4,577,425	23 1	741,965	1 19	771,915	74	342,925	1 8	1,548,350	8 2	59	6	Great Western.
Great Eastern	9,054,100	731,437	2 00	8 7	7 0	818,570	9 0	9 0	17 7	16 0	767,100	8 4	8 4	26 1	24 4	1,481,740	16 4	348,535	91	418,960	1 32	156,240	7	695,983	7 7	56	5	Great Eastern.
South Eastern	4,741,100	422,405	2 00	8 8	8 6	381,965	7 8	7 8	16 4	12 4	590,200	12 6	5 6	22 0	18 0	714,745	14 7	209,500	.....	120,000	.....	134,580	1 1	653,720	13 4	48	5	South Eastern
London, Chatham and Dover	3,119,750	209,957	1 6	8 2	8 7	197,910	6 8	6 8	15 0	10 5	218,495	7 5	7 5	22 5	17 0	449,585	15 4	149,240	70	68,155	1 12	74,595	1 5	328,175	11 2	53	Nil	London, Chatham and Dover.
Caledonian	3,951,750	450,465	1 6	6 9	6 9	668,295	8 5	8 5	14 4	14 4	617,175	7 9	7 9	22 3	22 3	1,067,220	14 1	338,640	1 83	353,765	1 43	119,725	1 7	488,435	5 8	52	4	Caledonian.
Glasgow and South Western	1,878,025	184,570	1 60	6 5	6 5	232,660	8 2	8 2	14 7	14 7	237,850	8 4	8 4	24 1	24 1	444,020	20 0	114,600	.....	228,645	.....	69,385	2 5	211,275	7 4	54	4	Glasgow and South Western.
North British	6,195,800	473,900	1 60	7 8	7 8	710,550	10 8	10 8	16 6	16 6	570,745	9 7	9 7	29 1	24 3	1,263,805	19 0	210,715	1 08	491,565	1 25	110,255	1 7	368,295	5 6	71	4	North British.
Great South. and West. Ry. of Ireland	2,142,057	167,456	.....	.....	.....	139,595	6 4	6 4	14 9	.....	178,045	8 7	8 7	29 1	.....	504,876	25 0	89,870	.....	91,767	.....	44,245	1 7	141,185	5 6	70	7	Great Southern and Western of Ireland.
London, Brighton and South Coast	5,471,410	347,715	1 60	10 0	7 1	474,620	8 7	8 7	15 7	14 0	577,215	9 8	6 8	25 5	20 8	857,570	15 8	334,175	90	140,795	1 08	114,670	2 1	631,429	11 5	51	4	London and Brighton and South Coast.
London and South Western	8,230,125	716,110	1 60	8 0	8 7	624,690	7 6	7 6	16 3	12 0	371,020	4 5	4 5	29 8	16 7	1,471,190	17 9	481,591	88	264,890	1 03	150,860	1 8	643,995	7 8	59	5	London and South Western
Average, 14 Railroads	.....	.....	.....	7 2	7 7	.....	8 0	8 0	15 8	14 5	.....	7 8	7 8	13 6	22 1	.....	17 9	.....	.....	.....	.....	.....	1 9	.....	.....	.....	.....	Average, 14 Railroads.
UNITED STATES, 1884																											UNITED STATES, 1884.
Boston and Albany	3,785,877	1,350,294	4 42	11 4	4 1	402,575	6 9	3 5	18 4	7 2	391,737	6 8	3 1	25 1	12 4	1,344,741	24 3	173,632	1 14	400,785	1 09	119,344	2 1	322,870	9 0	71	8	Boston and Albany.
Boston and Lowell	2,074,243	1,264,496	4 78	12 7	4 2	163,865	6 4	3 7	20 1	7 7	128,554	5 0	4 6	26 3	12 1	456,991	25 0	99,277	1 47	51,734	2 05	51,734	2 3	113,041	5 5	72	5	Boston and Lowell.
Boston and Maine	2,980,874	2,134,517	4 48	11 3	4 0	345,420	6 5	3 3	17 8	6 9	116,698	5 6	4 2	23 4	11 1	444,291	21 4	90,871	1 01	55,827	1 24	57,927	2 8	146,757	6 1	67	8	Boston and Maine.
Boston and Providence	1,561,119	1,126,488	4 13	8 8	3 1	93,142	6 9	3 5	15 7	6 3	82,674	6 7	3 0	22 4	11 3	364,026	25 9	97,695	1 77	39,124	1 80	43,726	2 5	104,852	7 6	70	7	Boston and Providence.
Old Colony	2,914,257	2,278,150	4 42	9 6	3 8	216,962	7 3	3 7	16 8	7 1	142,437	4 8	3 6	21 6	10 3	853,044	28 4	147,112	1 26	94,459	1 61	69,939	2 1	199,361	6 8	72	7	Old Colony.
New York, New Haven and Hartford	4,828,240	4,504,448	4 39	9 3	3 3	259,758	5 8	2 9	15 1	5 9	155,425	3 2	2 4	18 3	8 3	1,097,891	22 7	258,546	1 35	103,257	82	102,486	2 1	325,515	6 7	71	10	New York, New Haven and Hartford.
New York, Lake Erie and Western	12,079,638	11,242,980	2 46	10 3	6 9	1,019,438	8 4	4 2	18 7	11 1	491,902	4 1	2 1	22 8	15 2	1,777,243	14 8	574,076	1 49	743,089	48	186,695	1 4	391,934	2 5	70	Nil	New York, Lake Erie and Western.
New York Central and Hudson River	17,849,313	14,956,987	3 20	11 0	5 5	1,398,982	7 5	3 8	19 3	9 3	865,164	4 9	3 7	22 4	13 0	3,088,032	17 3	509,178	1 31	1,178,034	54	170,645	1 0	999,866	5 6	70	8	New York Central and Hudson River.
Pennsylvania—Pennsylvania Division	17,575,106	14,094,744	1 60	6 2	6 2	1,257,95	7 2	3 6	14 4	9 8	1,497,444	8 5	6 4	21 9	16 2	3,225,664	18 8	443,642	1 27	1,089,021	57	202,109	1 1	466,271	2 7	68	8	Pennsylvania—Pennsylvania Division.
Baltimore and Ohio	6,147,666	5,349,240	1 60	5 6	6 5	456,697	7 4	3 7	12 3	9 2	624,146	10 1	7 6	23 0	16 8	1,385,176	22 6	209,387	.....	709,029	.....	.....	.....	.....	.....	56	10	Baltimore and Ohio.
Average, 10 Railroads	.....	.....	.....	9 6	4 6	.....	7 1	3 6	16 7	8 0	.....	6 1	4 6	22 8	12 7	.....	21 8	.....	.....	.....	.....	.....	1 9	.....	.....	.....	.....	Average, 10 Railroads.

TABLE No. 24.

Analysis of the Traffic on the Principal Railroads of the United Kingdom and of the United States.

NAME OF RAILROAD	TRACKS, EXCLUSIVE OF SIDINGS AND TURN-OUTS: Total Length of Line Operated	Single Track	Two Tracks.	Three Tracks.	Four Tracks.	Total, all reduced to Single Track	PASSENGER TRAFFIC. EXCLUSIVE OF SEASON TICKETS: Total Number.	Total Receipts in Pounds Sterling.	Average Amount Received from each Passenger	Average Mileage of each Passenger	Total Passengers carried 1 Mile.	SEASON TICKETS: Total Number.	Total Receipts in Pounds Sterling.	Total Mileage at the rate of 600 Miles for each Pound Sterling.	TOTAL PASSENGER MILEAGE.: Total.	Per Mile of Single Track.	Average Fares per Mile. 1st 2d and 3d Class in United Kingdom, and 1st only in United States	Total Number of Tons Transported	FREIGHT TRAFFIC. Total Receipt from Freight.	Average Receipt or Charge from each Ton	Average Mileage of each Ton.	TOTAL TON MILEAGE.: Total.	Per Mile of Single Track	Average Charge per Ton on all Freight Trains—portion. Per Mile.	NAME OF RAILROAD.
	Miles.	Miles.	Miles.	Miles.	Miles.	Miles.		£	d	Miles			£				£ d Dollars.		£	d Dollars.	Miles				
UNITED KINGDOM 188.								£																	UNITED KINGDOM, 1883.
Great Northern	768	768	604	....	.....	1 372	23 701 706	1 147 375	11 5	10	426 359 250	16 098	83 588	50 152 800	286 512 050	208 80	$6 — 1 433	9 429 128	2 936 741	7	1 68	7 1	7 2 479 552	3 602 299	$0 02	Great Northern.
North Eastern	1 554	1 554	1 114	....	.....	2 548	35 473 469	1 728 142	11	9	594 797 262	17 978	94 889	56 933 400	351 184 052	137 861	0 0 — 1 404	30 756 125	4 882 147	5	2 78	3 2	9 1 753 091 29	455 118	0 02	North Eastern.
Midland	1 481	1 381	1 091	....	.....	2 384	31 501 707	1 801 079	14 0	12	571 444 274	17 458	101 196	60 717 600	442 151 874	185 472	0 0 — 1 553	25 440 595	5 072 988	4	3 96	4 8	1 222 309 49	512 714	0 02	Midland.
London and North Western	1 758	1 758	1 337	....	.....	3 290	55 894 947	3 216 589	14 0	12	964 617 364	104 467	138 953	83 131 800	745 749 134	226 777	0 0 — 1 553	45 754 500	6 290 081	4	2 75	4 2	1 541 777 440	474 932	0 02	London and North Western.
Great Western	2 208	2 208	1 104	....	.....	3 377	49 578 604	2 883 122	14 0	12	780 922 720	24 478	80 172	48 103 200	742 025 920	219 711	0 0 — 1 553	24 24 — 3 29	4 043 575	4	3 82	4	1 042 188 24	308 7 —	0 02	Great Western.
Great Eastern	1 040	1 040	850	....	.....	1 760	62 442 193	1 440 281	5 5	4 7	480 697 886	18 888	143 594	86 156 400	481 451 286	273 580	0 0 — 1 553	7 182 111	1 556 755	4	1 4	5 2	10 964 4 85	231 140	0 2	Great Eastern.
London and South Western	721	721	591	....	.....	1 222	24 886 691	1 553 561	10 0	9 3	520 031 506	104 963	143 924	86 354 400	406 385 906	332 510	0 1 — 1 553	4 672 750	802 758	3	1 18	7 3	216 647 126	177 231	0 02	London and South Western
London, Chatham and Dover	109	109	110	....	.....	219	2 967 252	207 557	5 7	4 9	145 496 856	77 913	84 201	50 520 600	195 947 106	894 280	0 0 — 1 553	2 257 214	387 —	3	1 —	5 8	68 540 969	312 506	0 2	London, Chatham and Dover.
Caledonian	877	877	662	....	.....	1 539	17 085 464	805 474	11 4	9 8	166 751 448	8 067	48 383	28 980 800	195 732 248	146 150	0 0 — 1 553	15 205 500	1 880 054	2	1 60	3 6	456 161 700	340 673	0 02	Caledonian.
North Britain	1 046	1 046	419	....	.....	1 439	19 111 553	778 090	9 8	8 4	166 286 546	17 157	85 743	33 445 800	143 736 440	136 575	0 0 — 1 553	13 680 — 97	1 651 013	2	1 58	2 9	89 244 152	229 241	0 02	North Britain.
London, Brighton and South Coast	440	440	273	....	.....	676	41 296 067	1 232 482	7 7	6 4	264 220 672	15 105	172 008	103 204 800	368 004 192	544 — 28	0 0 — 1 553	2 544 971	542 — 2	4	1 42	3 8	112 158 — 84	165 — 4	0 02	London, Brighton and South Coast.
Total of all Railroads—United Kingdom	18 084	18 081	10 105	.....	.....	28 786	683 718 137	24 049 861	8 4	7 2	4 954 271 926	322 050	1 092 291	1 015 551 600	5 969 825 366	207 387	0 0 — 1 553	266 082 988	38 761 10	3	2 —	4	6 288 176 760	218 420	0 02	Total of all Railroads—United Kingdom.
UNITED STATES 1884.								Dollars.	Dollars.								1st Class Mileage		Dollars						UNITED STATES, 1884.
Boston and Albany	394	384	211	10	9	614	8 748 472	3 194 452	.....	.....	.....	.....	.....	.....	167 442 441	272 642	$0 01 — 1	1 447 717	14 480 —		1 25	112	674 247 457	662 686	$0 0109	Boston and Albany.
Boston and Lowell	550	550	40	.....	.....	681	4 959 043	1 301 314	.....	.....	.....	.....	.....	.....	161 483 056	295 971	0 02 — 1	1 477 708	1 773 950		1 20	80	2 142 729 091	3 924	0 0231	Boston and Lowell.
Boston and Maine	294	294	87	.....	.....	371	7 054 401	1 701 228	.....	.....	.....	.....	.....	.....	89 587 566	240 561	0 0192	958 642	1 045 664		1 14	47	43 137 744	165 821	0 0234	Boston and Maine.
Boston and Providence	68	68	44	.....	.....	112	5 482 767	1 046 546	.....	.....	.....	.....	.....	.....	85 260 708	392 863	0 01878	742 598	811 — 6		1 09	86	21 814 483	194 772	0 02817	Boston and Providence.
Old Colony	466	408	57	.....	.....	525	7 826 264	2 188 197	.....	.....	.....	.....	.....	.....	116 745 963	222 452	0 0187	1 647 048	1 741 803		1 05	86	57 879 872	110 285	0 01	Old Colony.
New York, New Haven and Hartford	567	265	175	.....	.....	493	7 782 214	4 918 460	.....	.....	.....	.....	.....	.....	206 677 775	516 694	0 0196	2 181 — 5	2 440 — 14		1 12	58	125 745 893	314 364	0 0197	New York, New Haven and Hartford
New York, Lake Erie and Western	1 029	1 029	440	.....	.....	1 472	5 485 699	4 677 871	.....	.....	.....	.....	.....	.....	169 789 245	114 982	0 0216	11 — 71	15 775 — 44		1 42	162	1 704 946 319	1 216 913	0 0098	New York, Lake Erie and Western.
New York Central and Hudson River	993	993	514	317	299	2 123	11 037 969	7 593 813	.....	.....	.....	.....	.....	.....	887 829 886	182 860	0 0104	10 212 418	10 844 986		1 61	193	1 970 087 115	927 973	0 0083	New York Central and Hudson River.
Baltimore and Ohio	853	.....	.....	.....	.....	.....	.....	.....	.....	38 5	.....	.....	.....	.....	69 395 167	.....	.....	.....	.....		.....	191	847 879 421	.....	.....	Baltimore and Ohio.
Pennsylvania Railroad—																									Pennsylvania Railroad—
Pennsylvania Division	1 438	1 358	386	106	29	1 879	10 697 474	6 115 423	0 57	24 0	.....	.....	.....	.....	244 710 876	130 235	0 0244	21 674 163	24 578 — 130		1 14	124	2 990 892 867	1 594 498	0 00740	Pennsylvania Division.
United Railroads of New Jersey Div.	.....	442	134	.....	.....	576	12 064 050	5 699 245	0 45	19 6	.....	.....	.....	.....	248 783 706	441 927	0 0240	9 144 735	7 833 914		86	60	552 423 171	959 070	0 01365	United Railroads of New Jersey Div.

TABLE No. 25.

Analysis of Train Service, etc., on the Principal Railroads of the United Kingdom and of the United States.

United Kingdom, 1883.	Total Length of Lines Operated. Miles.	All reduced to Single Track. Miles.	Passenger Train Mileage.		Passenger Mileage.		Freight Train Mileage.		Ton Mileage.		Total Freight and Passenger Train Mileage per Mile of Single Track.
			Total.	Per Mile of Single Track.	Total.	Average per Train-Mile.	Total.	Per Mile of Single Track.	Total.	Average per Train-Mile.	
Great Northern	768	1 372	8 091 256	5 890	286 512 050	35.4	8 238 177	6 005	502 973 520	61.5	11 895
North Eastern	1 534	2 548	9 681 906	3 800	359 184 052	37.1	14 763 232	5 798	1 159 691 280	78.5	9 598
Midland	1 381	2 384	13 105 400	5 493	432 151 874	32.9	19 981 855	8 382	1 222 309 920	61.0	13 875
London and North Western	1 793	3 190	18 931 111	5 935	745 749 134	39.4	19 395 461	6 080	1 511 779 440	78.0	12 015
Great Western	2 268	3 377	14 298 157	4 234	642 025 920	44.9	16 047 939	4 752	1 042 489 200	65.0	8 986
Great Eastern	1 049	1 599	8 486 358	5 307	386 454 286	45.5	5 334 027	3 336	369 094 320	69.2	8 643
South Eastern	370	699	4 930 330	7 053	326 660 480	66.3	1 416 949	2 027	121 009 920	85.4	9 080
London, Chatham and Dover	160	309	3 152 677	10 202	195 917 102	62.2	678 173	2 195	60 534 960	89.5	12 397
Caledonian	877	1 339	5 232 096	3 908	195 721 238	37.4	6 617 610	4 942	456 161 700	68.9	8 850
Glasgow and South Western	332	571	2 230 838	3 907	92 966 400	41.7	2 294 680	4 020	154 419 120	67.0	7 927
North British	1 006	1 419	5 032 790	3 547	193 726 340	38.5	6 428 496	4 537	396 243 120	61.3	8 084
Great Southern and Western of Ireland	478	673	1 678 691	2 494	66 844 142	39.8	1 164 993	1 730	86 676 000	74.4	4 224
London, Brighton and South Coast	403	676	6 521 451	9 648	369 993 492	56.7	1 427 306	2 111	130 318 080	91.0	11 759
London and South Western	721	1 222	8 001 548	6 548	406 375 606	50.8	3 045 365	2 492	216 649 920	71.0	9 040
United States, 1884.											
Boston and Albany	384	614	1 949 970	3 176	167 402 441	85.9	3 010 215	4 903	374 347 455	124.3	8 079
Boston and Lowell	550	590	1 305 150	2 212	61 343 056	47.0	610 162	1 031	59 427 991	97.4	3 243
Boston and Maine	204	271	1 289 545	4 759	89 587 566	69.5	540 028	1 993	44 937 544	83.2	6 752
Boston and Providence	68	112	727 348	6 494	55 200 708	76.0	246 298	2 200	21 814 483	88.6	8 694
Old Colony	468	525	1 629 344	3 104	116 745 901	71.6	799 610	1 523	57 899 872	72.4	4 627
New York, New Haven and Hartford	265	400	2 365 746	5 914	206 677 775	87.3	1 461 939	3 655	125 743 803	86.0	9 569
New York, Lake Erie and Western	1 029	1 475	3 791 414	2 571	169 599 245	44.8	7 513 873	5 098	1 794 946 519	239.0	7 669
New York Central and Hudson River	993	2 123	6 425 258	3 027	387 829 886	60.4	10 027 577	4 700	1 970 087 115	196.5	7 727
Pennsylvania—Pennsylvania Division	1 358	1 879	5 775 873	3 074	243 298 222	42.1	15 015 905	8 000	3 082 499 986	205.0	11 074
Pennsylvania—United Railroads of New Jersey Division	443	576	4 168 381	7 237	248 789 706	60.0	4 240 545	7 362	552 423 171	130.0	14 599

TABLE No. 26.

Comparison of the Average Annual Cost of Motive Power on Five of the Principal Railroads of England and the United States, for the Years 1882, 1883 and 1884.

Name of Railroad.	Total Operating Expenses, less Steamboat Expenses, Taxes and Duties.	Fuel.			Wages of Engine-drivers and Firemen.			Repairs and Renewals of Locomotives.				Total percentage of Fuel; wages of Engine-drivers & Firemen; and Repairs and Renewals of Locomotives to Operating Expenses.	
		Amount actually Paid.	Percentage on Operating Expenses.		Amount Paid.	Percentage on Operating Expenses.		Amount actually Paid.	Percentage on Operating Expenses.			Amount actually Paid.	All reduced to English Prices.
			Amount actually Paid	All reduced to English prices ($1.60 per ton of 2 240 pounds).		Amount actually Paid.	All reduced 50 per cent. to English Prices.		Amount actually Paid.	One-half of Cost of Repairs being Labor, which is here reduced 50 % to equalize it with the English.			
England.													
a Great Northern	£1 998 954	£132 495	6.6	6.6	£177 270	8.9	8.9	£135 734	6.8	6.8		22.3	22.3
a North Eastern	3 253 949	219 879	6.8	6.8	317 567	9.8	9.8	405 114	12.5	12.5		29.1	29.1
a Midland	3 607 350	227 303	6.3	6.3	425 629	11.8	11.8	351 702	9.8	9.8		27.9	27.9
a London and North Western	4 845 722	271 191	5.6	5.6	453 566	9.4	9 4	364 411	7.5	7.5		22.5	22.5
a c Great Western (2½ years)	3 512 520	195 945	5.6	5.6	315 892	9.0	9.0	372 298	10.6	10.6		25.2	25.2
Average			6.2			9.8	9.8		9.4	9.4		25.0	25.0
United States.													
b Boston and Albany	$5 353 344	$702 442	13.1	4.2	$406 688	7.4	3.7	$417 792	7.8	5.9		28.3	13.8
b New York, New Haven and Har.	4 205 166	429 977	10.2	4.2	252 301	6.0	3.0	152 469	3.6	2.7		19.8	9.9
b New York, Lake Erie and W.	13 248 239	1 370 670	10.3	6.9	1 114 471	8.4	4.2	632 474	4.8	3.6		23.5	14.5
b New York Central and H. R.	18 345 765	2 224 221	12.1	6.0	1 356 155	7.4	3.7	1 028 562	5.6	4.2		25.1	14.3
a Penn. R. R.—Penn. R. R. Div.	17 383 519	1 109 874	6.4	6.4	1 237 749	7.1	3.5	1 479 675	8.5	6.4		22.0	16.3
Average			10.4	5.5		7.3	3.6		6.1	4 6		23.7	13.8

a This data taken from reports of the company.
b This data taken from report of the Railroad Commissioners of New York and Massachusetts.
c Returns for the six months ending June 30th, 1884, could not be obtained, consequently they are omitted.
All charges for steamboat expenses have been deducted from the operating expenses.

The English roads were selected as being five of the roads doing the heaviest and largest traffic in the United Kingdom. The American five roads were selected from the Eastern system of roads, with the exception of the New York, New Haven and Hartford, built through a country more broken physically than that in which the English roads are located.

From this table, it appears that the greater cost in motive power of operating the English over the American railroads, as shown by Table 23, is not an accidental coincidence of one year, but is fully sustained by three years' working on five of the principal railroads in each country.

Supposing that the longer ton and passenger haulage of the above American lines influenced the result in their favor, the same calculation on the same basis, except the taxes are included in operating expenses, has been made for 1884, on the following short railroads of Massachusetts, where the average haulage is very short.

TABLE No. 27.

Name of Railroad.	Percentage on Operating Expenses, all Reduced to the Price Paid in England.			
	Coal.	Wages.	Repairs and Renewals of Locomotives.	Total.
Boston and Lowell.....	4.2	3.7	4.7	12.6
Boston and Maine.....	4.0	3.3	4.2	11.5
Boston and Providence.....	3.1	3.5	5.0	11.6
Old Colony.....	3.3	3.7	3.6	10.6
Average.....	3.6	3.6	4.4	11.6

The results of the workings of these small and short railroads are still more favorable to the American system.

As an additional check upon the cost of motive power on the American railroads, Table No. 28 was compiled. This contains an analysis of the cost of motive power on sixteen of the principal railroads of Massachusetts for the years 1883 and 1884. This table fully confirms the previous deductions, though this is hardly fair to the American railroads, as most of the roads in this table operate short lines and do small business,

consequently they cannot afford to keep up an efficient staff and have such thorough management as the large railroads of America or England. Moreover, as their consumption is small, they cannot buy their coal and other supplies on such advantageous terms as the larger consumers.

Cost per Train Mile.

In order to meet the views of those accustomed to compare the cost of train miles, Table No. 29 was made. The English make no separate return of the cost of motive power in freight and passenger traffic, so, for this table it was necessary to add the two together and take the aggregate train mileage. In consequence of the size of the trains varying so much in the two countries; the variation between different roads of the proportion between passenger and freight trains, which makes a proportionate variation in the cost of operating; and the absence of all official data from the English roads regarding their ton and passenger mileage; these deficiencies, in the opinion of the author, make this table less reliable than the previous ones, where the cost of motive power and other expenses were compared, by taking their percentage to the total operating expenses; for in roads constructed in similar countries, the different items of operating expenses should bear a close proportionate percentage to each other.

In order to be safe beyond all question, the Massachusetts railroads have been largely used for comparison. These have no advantage whatever over the English roads, being with one exception all short lines with short haulage, located in a rough and broken country. In every respect they are more unfavorably situated for cheap working than the average of the English roads. Moreover, the Reports of the Railroad Commissioners of Massachusetts are as reliable as those of the English Board of Trade, and much more thorough and complete—these give very reliable data for comparison.

In the repairs of locomotives, in the English cost there has been included nothing except the actual material and labor used in the repairs, while in the American cost is included superintendence, cost and repairs of tools, shop, etc. This would make the difference in the cost still greater if these items were added to the English cost.

In Table No. 29, after reducing the cost down to the English standard of prices, the average of six American railroads cost in motive power

TABLE No. 28.

Analysis of the Cost of Motive Power on Sixteen of the Principal Railroads of Massachusetts in the Years 1883 and 1884.

	Length of Road Operated. Miles.	Total Operating Expenses, exclusive of Taxes.	FUEL. Total Amount Paid.	Price per Ton of 2 240 lbs.	Tons of Coal.	Cords of Wood.	Total Tons of Coal, 1½ Cords of Wood = 1 Ton of Coal.	FUEL. Percentage on Operating Expenses. Amount Paid.	FUEL. Percentage on Operating Expenses. Reduced to $1.60 per Ton of 2 240 lbs.	WAGES OF ENGINE-DRIVERS AND FIREMEN. Total Amount Paid.	WAGES OF ENGINE-DRIVERS AND FIREMEN. Percentage on Operating Expenses. Amount Paid.	WAGES OF ENGINE-DRIVERS AND FIREMEN. Percentage on Operating Expenses. Reduced 50 % to Equalize with English Prices.	REPAIRS AND RENEWALS OF LOCOMOTIVES. Total Amount Paid.	REPAIRS AND RENEWALS OF LOCOMOTIVES. Percentage on Operating Expenses. Amount Paid.	REPAIRS AND RENEWALS OF LOCOMOTIVES. Percentage on Operating Expenses. Reduced 25 % to Equalize Wages with English.	Percentage of Cost of Fuel, Wages of Engine-drivers and Firemen, and Repairs and Renewals of Locomotives on Total Operating Expenses. Amount Actually Paid.	Percentage of Cost of Fuel, Wages of Engine-drivers and Firemen, and Repairs and Renewals of Locomotives on Total Operating Expenses. All Reduced to the English Standard of Prices.	Dividends Earned. Per Cent.
Boston and Albany, 1883	374	$5 668 718	$795 383	$5.40	145 700	2 348	.....	....	....	$414 643	....	....	$434 726	....	....	....	....	8.2
" 1884	384	5 263 027	659 592	4.40	148 200	2 480	.....	....	....	402 376	....	....	391 757	....	....	....	....	8.1
" average	....	5 465 873	727 488	4.90	.....	.....	.....	13.3	4.4	408 510	7.4	3.7	413 242	7.6	5.7	28.3	13.8	....
Fitchburg, 1883	189	2 045 547	311 785	5.09	60 842	462	.....	....	....	176 329	....	....	125 710	....	....	....	....	6.4
" 1884	189	1 979 609	296 577	4.62	63 845	518	.....	....	....	184 964	....	....	133 792	....	....	....	....	5.4
" average	....	2 012 578	304 181	4.86	.....	.....	.....	15.1	5.0	180 647	8 9	4.5	129 751	6.4	4.8	20.4	14.3	....
Boston and Lowell, 1883	170	1 431 240	198 069	5.02	38 378	397	.....	....	....	115 553	....	....	90 183	....	....	....	....	6.5
" 1884	550	1 961 252	263 490	4.78	43 020	18 203	.....	....	....	153 968	....	....	128 354	....	....	....	....	5.7
" average	....	1 696 246	230 780	4.90	.....	.....	.....	13.6	4.4	134 760	7.9	3.9	109 269	6.4	4.8	27.9	13.1	....
Boston and Maine, 1883	204	1 955 496	252 602	4.94	50 908	381	.....	....	....	128 386	....	....	106 743	....	....	....	....	8.2
" 1884	204	1 954 117	234 517	4.48	52 251	221	.....	....	....	135 419	....	....	116 698	....	....	....	....	8.2
" average	....	1 954 807	243 560	4 71	.....	.....	.....	12.5	4.2	131 903	6.7	3.4	111 721	5.7	4.3	24.9	11.9	....
Eastern, 1883	285	2 239 721	294 056	4.63	59 670	5 813	.....	....	...	166 284	....	....	202 854	....	....	....	....	4.0
" 1884	285	2 235 325	284 321	4.46	60 477	4 886	.....	....	....	170 715	....	....	174 921	....	....	....	....	3.8
" average	....	2 237 523	289 189	4.55	.....	.....	.....	12.9	4.5	168 499	7.5	3.7	188 888	8.4	6.3	28.8	14.5	....
Boston and Providence, 1883	68	1 220 149	120 790	4.92	24 574	.....	.....	....	....	87 327	....	....	84 568	....	....	....	....	8.2
" 1884	68	1 259 287	120 488	4.59	26 222	.....	.....	....	....	95 132	....	....	82 674	....	....	....	....	8.2
" average	....	1 239 718	120 639	4.76	.....	.....	.....	9.7	3.4	91 230	7.4	3.7	83 621	6.7	5.0	23.8	12.1	....
Old Colony, 1883	468	3 057 361	310 465	5.20	59 281	633	.....	....	....	215 874	....	....	194 193	....	....	....	....	6.0
" 1884	468	2 744 914	278 140	4.47	62 013	339	.....	....	....	215 302	....	....	142 436	....	....	....	....	7.0
" average	....	2 901 138	294 303	4.84	.....	.....	.....	10.1	3.4	215 588	7.4	3.7	168 315	5.8	4.3	23.3	11.4	....
Providence and Worcester, 1883	50	793 959	76 198	5.59	13 642	.....	.....	....	....	56 557	....	....	29 680	....	....	....	....	8.5
" 1884	50	819 030	75 143	4.85	15 520	.....	.....	....	....	55 459	....	....	59 204	....	....	....	....	6.8
" average	....	806 495	75 670	5.22	.....	.....	.....	9.4	3.0	56 008	6.9	3.5	44 442	5.5	4.1	21.8	10.6	....
New Haven and Northampton, 1883	173	564 788	68 663	4.60	14 927	.....	.....	...	....	32 635	....	....	47 717	....	....	....	....	Nil.
" 1884	173	541 613	64 243	4.00	16 061	.....	.....	....	....	36 903	....	....	35 245	....	....	....	....	Nil.
" average	....	553 200	66 453	4.30	.....	.....	.....	12.0	4.5	34 769	6.0	3.0	41 481	7.5	5.6	25.5	13.1	....
New London Northern, 1883	121	462 904	74 280	5.00	13 681	1 769	.....	....	....	30 431	....	....	69 329	....	....	....	....	6.0
" 1884	121	318 974	54 692	4.20	11 451	2 338	.....	...	....	26 352	....	....	18 675	....	....	....	....	6.0
" average	....	390 939	64 486	4.60	.....	.....	.....	16.5	5.8	28 392	7.3	3.6	44 002	11.3	8.5	35.1	17.9	....
New York and New England, 1883	378	3 036 469	475 197	3.59	132 292	.....	.....	....	....	233 481	....	....	332 086	....	....	....	....	Nil.
" 1884	378	2 816 610	415 464	3.82	108 623	.....	.....	....	....	224 220	....	....	331 141	....	....	....	....	Nil.
" average	....	2 926 540	445 330	3.71	.....	.....	.....	15.2	6.6	228 851	7.8	3.9	331 614	11.4	8.6	34.4	19.1	....
Cheshire, 1883	64	454 556	102 386	6.23	9 822	9 902	.....	....	....	36 045	....	....	27 367	....	....	....	....	2 8
" 1884	64	386 635	98 950	6.08	8 927	11 025	.....	...	....	31 385	....	....	30 574	....	....	....	....	3.5
" average	....	420 596	100 668	6.16	.....	.....	.....	23.9	6.2	33 715	8.0	4.0	28 970	6 9	5.2	38.8	15.4	....
Connecticut River, 1883	80	588 563	92 222	6.00	13 651	2 713	.....	....	....	41 846	....	....	30 209	....	....	....	....	9.5
" 1884	80	568 224	88 985	5.68	13 886	2 672	.....	....	....	41 470	....	....	33 906	....	....	....	....	9.1
" average	....	578 394	90 604	5.84	.....	.....	.....	15.7	4.3	41 658	7.2	3.6	32 058	5.6	4.2	28.5	12.1	....
Norwich and Worcester, 1883	66	453 420	60 617	4.80	12 599	40	.....	....	....	37 327	....	....	29 425	....	....	....	....	8.6
" 1884	66	418 687	48 689	4.30	11 234	60	.....	....	....	35 544	....	....	28 030	....	....	....	....	8.1
" average	....	436 054	54 653	4.55	.....	.....	.....	12 6	4.4	36 436	8.4	4.2	28 728	6.6	5.0	27.6	13.6	....
Worcester, Nashua and Rochester, 1883	94	471 520	89 099	5.95	14 970	.....	.....	....	....	29 700	....	....	41 432	....	....	....	....	3.3
" 1884	94	444 320	72 752	5.16	14 094	.....	.....	....	....	29 980	....	....	39 105	....	....	....	....	2.6
" average	....	457 920	80 926	5.55	.....	.....	.....	17.7	5.2	29 840	6.5	3.3	40 269	8.8	6.6	33.0	15.1	....
New York, New Haven and Hartford, 1883	265	4 205 166	429 977	4.34	98 682	133	.....	....	....	252 301	....	....	152 469	....	....	....	....	10.2
" " 1884	265	4 205 166	429 977	4.19	102 654	.....	.....	....	....	252 301	....	....	152 469	....	....	....	....	10.1
Average of three years, 1882, 1883 and 1884	....	4 205 166	429 977	4.27	.....	.....	.....	10.2	4.2	252 301	6.0	3.0	152 469	3.6	2.7	19.8	9.9	....
Total	3 240	56 566 467	7 237 919	.....	1 522 106	67 333	1 566 994	....	....	4 146 209	....	....	3 897 672	....	....	....	....	....
Average—16 Massachusetts Railroads	....	.....	.....	4.62	.....	.....	.....	12.8	4.4	.....	7.4	3.7	.....	6.9	5.2	27.1	13.3	....
" 14 Railroads of United Kingdom, from Table 23	....	.....	.....	.....	.....	.....	.....	7.2	7.2	.....	8.6	8.6	.....	7.8	7.8	23.6	23.6	....

1½ cords of wood calculated to equal one ton of coal.

$0.021, or 14 per cent. less per train mile than the average cost of six English railroads, notwithstanding that the average load of the American train was more than double in tons, and over 50 per cent. more in passengers. This corresponds very closely with the result obtained by calculating upon the percentage basis.

The London and North Western, of England, and the Boston and Albany, of the United States, have each about double the ton mileage to that of their passenger mileage, consequently they offer a good basis for comparison. The motive power (the prices being reduced to the English rates) costs the same per train mile. The American road, however, averaged 60 per cent. more load on their freight trains, and more than double the load on their passenger trains.

The London and North Western Railway had in 1883 a total passenger train mileage of 18 931 111. Supposing that they carried the same average load as the Boston and Albany did in 1884, *i. e.*, 85.9, this will give a total passenger mileage of $18\ 931\ 111 \times 85.9 = 1\ 626\ 182\ 435$. The total receipts from all classes of passengers were £3 355 142; dividing the total passenger mileage as above into this, gives .495d. as the average fare per mile paid by each passenger, first, second and third-class included.

In 1883 the London and North Western Railway ran 19 395 461 freight train miles. Supposing that they transported the same average load as the Boston and Albany did in 1884, *i. e.*, 124.3 tons, this will give a total ton mileage of $19\ 395\ 461 \times 124.3 = 2\ 410\ 855\ 802$; dividing this into £6 299 081, the total amount received from freight traffic, gives .627d. as the average price paid per ton per mile.

These equated prices are less than one-half of the actual or ruling prices.*

The Great Eastern, of England, and the Boston and Lowell, of the United States, also offer a good basis for comparison, as each have about equal passenger mileage to their ton mileage. Reducing all to the English standard of prices, the American road is operated at $0.026, or 19 per cent. less cost per train mile than the English, while the freight load of the American train is 40 per cent. larger and the passenger load five per cent. greater.

* The present freight charge by the London and North Western Railway from Manchester to Liverpool, 29 miles, is 9s. 2d. or $2.20 per ton of 2 240 pounds for goods in bales and cases, being about 7.6 cents per ton per mile.

TABLE No. 29.

Cost of Motive Power per Train Mile on Six English and Six American Railroads.

NAME OF RAILROAD.	TOTAL MILEAGE.			AVERAGE LOAD OF TRAIN.		AVERAGE MILEAGE.	
	Train. Passenger and Freight.	Passenger. (For English Railroads calculated as stated on page 17.)	Ton. (For English Railroads calculated as stated on page 16.)	Passenger. Number of Passengers Per Mile of Passenger Train.	Freight. Number of Tons Per Mile of Freight Trains.	Passenger.	Ton.
ENGLAND, 1883.							
Great Northern	16 329 433	288 069 860	502 973 520	35.6	61.5	.....	53
North Eastern	24 445 138	363 647 121	1 159 691 280	37.6	78.5	.....	29
Midland	33 087 255	436 338 084	1 222 309 920	33.3	61.	.....	48
London and North Western	38 326 572	744 151 164	1 511 779 440	39.3	78.	.....	42
Great Western	30 346 096	642 205 320	1 042 489 200	44.9	65.	.....	43
Great Eastern	13 820 385	382 764 707	369 094 320	45.1	69.2	.....	52
Average	.....	.....	.....	39.3	68.9	.....	44.5
UNITED STATES, 1884.							
Boston and Albany	4 960 185	167 402 441	374 347 455	85.9	124.3	.....	112
New York, New Haven and Hartford	3 827 685	206 677 775	125 743 803	87.3	86.	.....	58
New York, Lake Erie and Western	11 305 287	169 599 245	1 794 946 519	44 8	239.	.....	162
New York Central and Hudson River	16 452 835	387 829 886	1 970 087 115	60.4	196.5	.....	193
Pennsylvania—Pennsylvania Railroad Division	20 791 778	244 710 876	2 996 892 567	42.2	205	.....	134
Boston and Lowell	1 915 312	61 343 056	59 427 991	47.	97.4	.....	40
Average	.....	.....	.....	61.3	158.	.....	116.5

TABLE No. 29.—*Continued.*

NAME OF RAILROAD.	FUEL.			WAGES OF ENGINE-DRIVERS AND FIREMEN.			REPAIRS AND RENEWALS OF LOCOMOTIVES.			COST OF FUEL, WAGES OF ENGINE-DRIVERS AND FIREMEN, AND REPAIRS AND RENEWALS OF LOCOMOTIVES, PER TRAIN MILE.	
	AMOUNT ACTUALLY PAID.		Per Train Mile. Coal reduced to $1.60 per ton of 2 240 lbs.	AMOUNT ACTUALLY PAID.		Per Train Mile. All American Wages Reduced 50 Per Cent. to Equalize Them with English Wages.	AMOUNT ACTUALLY PAID.		Per Train Mile. All American Wages Reduced 50 Per Cent. to Equalize Them with English Prices.		
	Total.	Per Train Mile.		Total.	Per Train Mile.		Total.	Per Train Mile.		Amount Actually Paid.	All Reduced to English Prices.
ENGLAND, 1883.											
Great Northern	$699 965	$0.043	$0.043	$903 050	$0.055	$0.055	$680 635	$0.042	$0.042	$0.140	$0.140
North Eastern	1 150 780	.047	.047	1 640 655	.067	.067	1 994 480	.082	.082	.196	.196
Midland	1 167 305	.035	.035	2 153 220	.065	.065	1 803 705	.054	.054	.154	.154
London and North Western	1 386 285	.036	.036	2 295 860	.060	.060	1 839 260	.048	.048	.144	.144
Great Western	1 017 880	.034	.034	1 584 205	.052	.052	1 855 190	.061	.061	.147	.147
Great Eastern	793 935	.057	.040	818 870	.060	.060	765 990	.056	.056	.173	.162
Average	.....	$0.042	$0.040	.....	$0.060	$0.060	.....	$0.057	$0.057	$0.159	$0.157
UNITED STATES, 1884.											
Boston and Albany	$659 592	$0.133	$0.048	$402 376	$0.081	$0.041	$391 757	$00.079	$0.059	$0.293	$0.148
New York, New Haven and Hartford	450 436	.118	.045	259 758	.068	.034	155 425	.040	.030	.190	.109
New York, Lake Erie and Western	1 242 989	.110	.078	1 019 438	.090	.045	491 902	.044	.033	.244	.151
New York Central and Hudson River	1 956 397	.119	.060	1 338 142	.081	.040	865 184	.053	.040	.253	.140
Pennsylvania—Pennsylvania Railroad Division	1 094 534	.053	.053	1 257 956	.060	.030	1 497 444	.072	.054	.185	.137
Boston and Lowell	263 490	.138	.046	153 967	.080	.040	128 354	.067	.050	.285	.136
Average	.....	$0.112	$0.054	.....	$0.077	$0.038	.....	$0.059	$0.044	$0.242	$0.136

In Tables 30 and 31, this method of calculation has been carried out more fully. In these tables can be seen at a glance what would have been the average receipts from each ton or passenger transported one mile, provided the train load had been the same as that carried by the American railroad with which it is compared. To facilitate comparison, the cost of motive power per train mile has been transferred from Table No. 29.

These equated prices are, without exception, much too low, averaging less than half the ruling or present prices. They are so much too small, that any argument based upon them would be absurd.

They show conclusively, that notwithstanding the average train mile costs in motive power less on the American railroads than on the English, yet the American average train load is much greater both in tonnage and passengers.

In order to make the comparison still more comprehensive, Table No. 32 was made. In this all the railroads of Massachusetts, 32 in number, embracing 2 852 miles of operated lines, are compared with all the railroads of England and Wales, embracing 13 340 miles of operated lines. Both systems for the years 1879 to 1884 inclusive, or for six years.

For want of time, the author has not been able to ascertain officially from each railroad included in Table 32, the exact cost of coal consumed. From data given in the previous tables it would be perfectly safe, in his judgment, to estimate the cost of coal consumed on the Massachusetts railroads at three times the price per ton of that consumed on the English and Welsh railroads; but, in order to be conservative, he has only estimated it at double cost. Nearly all the coal consumed on the Massachusetts railroads must be transported over 200 miles by rail or 400 miles by water; with but few exceptions, all the principal railroads of England and Wales run direct into the coal districts. A thorough calculation, based upon the actual prices paid by each railroad, would show the operating expenses of the Massachusetts railroad on the cost of fuel alone to be at least sixteen per cent., or two cents less per train mile than those shown in Table No. 32.

Table No. 32 shows that the average load of passenger trains per passenger train mile in Massachusetts in 1884, was 67 passengers.

In England and Wales in 1884, the total passenger-train mileage was 121 582 978 miles. Suppose that these averaged the same load as the Massachusetts trains did, this will give $121\,582\,978 \times 67 = 8\,146\,905\,526$

TABLE No. 30.

Showing what would have been the Receipts from each Passenger per Mile Transported, First, Second and Third-class included, on the six English Railroads named in Table No. 28, if the Train Load had been the same as that of the American Railroads named in that table.

	Average Load of Passenger Trains. Number of Passengers.	Great Northern.	North Eastern.	Midland.	London and North Western.	Great Western.	Great Eastern.	Cost of Motive Power Per Train Mile. All reduced to English Prices.
Total Passenger Train Mileage	....	8 091 256	9 681 906	13 105 400	18 931 111	14 298 157	8 486 358	.....
Total Receipts from Passengers	....	£1 230 963	£1 602 120	£1 904 275	£3 355 142	£2 963 292	£1 589 875	.....
Boston and Albany	85.9	.43d.	.46d.	.41d.	.50d.	.58d.	.52d.	$0.148
New York, New Haven and Hartford	87.3	.42d.	.45d.	.40d.	.49d.	.57d.	.51d	0.109
New York, Lake Erie and Western	44.8	.81d.	.88d.	.78d.	.95d.	1.11d.	1.00d.	0.151
New York Central and Hudson River	60.4	.60d.	.66d.	.58d.	.70d.	.82d.	.74d.	0.140
Pennsylvania—Pennsylvania Railroad Division	42.2	.86d.	.93d.	.82d.	1.08d.	1.19d.	1.04d.	0.137
Boston and Lowell	47.	.78d.	.85d.	.75d.	.90d.	1.06d.	.95d.	0.136
Cost of Motive Power per Train Mile	....	$0.140	$0.196	$0.154	$0.144	$0.147	$0.162	.....

TABLE No. 31.

Showing what would have been the Receipts from each Ton Transported one Mile on the Six English Railroads named in Table No. 29, if the Train Load had been the same as that of the American Railroads named in that Table.

	Average Load of Freight Train. Number of Tons.	Great Northern.	North Eastern.	Midland.	London and North Western.	Great Western.	Great Eastern.	Cost of Motive Power per Train Mile. All Reduced to English Prices.
Total Freight Train Mileage	.....	8 238 177	14 763 232	19 981 855	19 395 461	16 047 939	5 334 027	.....
Total Receipts from Freight	.....	£2 095 723	£4 832 047	£5 092 958	£6 299 081	£4 343 705	£1 537 893	.....
Boston and Albany	124.3	.49d.	.63d.	.49d.	.63d.	.52d.	.55d.	$0.148
New York, New Haven and Hartford	86.	.71d.	.91d.	.71d.	.91d.	.76d.	.80d.	0.109
New York, Lake Erie and Western	239.	.26d.	.33d.	.26d.	.33d.	.27d.	.29d.	0.151
New York Central and Hudson River	196.5	.31d.	.40d.	.31d.	.40d.	.33d.	.35d.	0.140
Pennsylvania—Pennsylvania Railroad Division	205.	.29d.	.38d.	.29d.	.38d	.32d.	.33d.	0.137
Boston and Lowell	97.4	.63d.	.81d.	.58d.	.80d.	.66d.	.68d.	0.136
Cost of Motive Power per Train Mile	.....	$0.140	$0.196	$0.154	$0.144	$0.147	$0.162	.....

passenger miles. The total receipts in 1884 from all classes of passengers on the railroads of England and Wales was £22 247 095; dividing this by their total passenger mileage gives .655d. as the average charge per mile traveled by first, second and third-class.

In England and Wales in 1884, the total freight-train mileage was 106 605 484. Supposing that they averaged the same load as the Massachusetts trains did, *i. e.*, 109 tons, this gives $106\,605\,484 \times 109 =$ 11 619 997 756 total ton mileage. The total receipts from freight in 1884 by the English and Welsh railroads was £31 973 111; dividing this by the total ton mileage gives .660d. as the average charge per ton per mile.

These equated charges, when taken together for freight and passenger traffic, are so much below the real prices, being less than half, that no arguments can be based upon them.

Even supposing the average train-loads to be equal in both countries, still the motive power on the English and Welsh railroads costs, on the average, twenty-five per cent. more than on the Massachusetts railroads per train mile, as shown by Table No. 32, notwithstanding the underestimate of the cost of coal consumed.

Locomotive Mileage.

The locomotive with compound engines, as perfected by Mr. F. W. Webb, has given very satisfactory results, and is being largely introduced on the London and North Western Railway.

Mr. Webb kindly furnished the author with the data given in Table No. 33.

"As regards the loads hauled, we may mention that the average train taken between Crewe and Euston by the 'Dreadnought' class, consists of twelve vehicles. On the 19th of March last, the 'Dreadnought' worked the 10 A. M. Scotch express from Euston to Carlisle, a continuous run of 300¼ miles, with an average load, including engine and tender, of 207 tons. On this journey the consumption of fuel averaged 29.2 pounds per mile, and the evaporation of water was 9.49 pounds per pound of coal. The train arrived at Rugby two minutes before time, left four minutes late, and arrived at Carlisle four minutes before time, the Shap incline—5½ miles of 1 in 75—being mounted in ten minutes, and the average running speed over the whole trip being 44.7 miles per hour. Neither the smoke-box or ash-pan was cleared out during the journey, nor was the fire cleaned. The engine steamed freely throughout the run, and the weight of ashes in the smoke box on the arrival at Carlisle was 40¼ pounds, and in the ash-pan 59¼ pounds. These are certainly most admirable results.

"On March 27th last, also, engine No. 504, 'Thunderer,' worked

TABLE No. 32.

Average Cost per Train Mile for Motive Power on the Aggregate Railroads of England and Wales, and of the State of Massachusetts, for the Years 1879, 1880, 1881, 1882, 1883 and 1884.

Year.		Total Length of Lines Operated. Miles.	Average Miles each Passenger was Transported	Average Miles each Ton was Transported.	Total Train Mileage.	Average Train Load. Passengers.	Average Train Load. Tons	Fuel.	Wages of Engine-drivers and Firemen.	Repairs and Renewals of Locomotives.	Total Average. Pence.	Total Average. Dollars.
1879	England and Wales.....	.....	.....	.....	.....	.....	.....	*b*	*b*	*b*	8.40	0.1680
"	Massachusetts.....	2 626	16	56	22 755 910	57	90	$0.048	*a* $0.034	$0.034	.....	.116
1880	England and Wales.....	.....	.....	.....	202 050 891	.....	.....	*b*	*b*	*b*	8.30	.1660
"	Massachusetts	2 626	16	56	24 075 392	63	98	$0.054	*a* $0.033	$0.042	.....	.139
1881	England and Wales.....	.....	.....	.....	208 517 549	.....	.....	*b*	*b*	*b*	8.34	.1668
"	Massachusetts	2 755	16	60	27 205 783	64	104	$0.061	*a* $0.034	$0.043	.....	.138
1882	England and Wales.....	.....	.....	.....	217 393 015	.....	.....	*b*	*b*	*b*	8.37	.1674
"	Massachusetts.....	2 778	16	59	29 052 800	66	107	$0.060	*a* $0.033	$0.046	.....	.139
1883	England and Wales.....	13 215	.....	.....	226 351 389	.....	.....	*b*	*b*	*b*	8.41	.1682
"	Massachusetts.....	2 783	15	60	31 150 823	66	107	$0.062	$0.034	$0.050	.....	.146
1884	England and Wales.....	13 340	.....	.....	228 972 559	.....	.....	*b*	*b*	*b*	8.49	.1698
"	Massachusetts.....	2 852	15	60	32 304 333	67	100	$0.056	$0.033	$0.045	.....	.134
AVERAGE.	England and Wales.....	.....	.....	.....	.....	.....	.....	*b*	*b*	*b*	8.39	.1678
"	Massachusetts	.....	.....	.....	.....	.....	.....	$0.057	$0.034	$0.043	.....	.134

NOTES.—*a* The author has no official data for these years. The amount given is the average cost for the years 1883 and 1884.

b The Board of Trade includes all these items under the general head of "Locomotive Power," which is included in the total average at the bottom of this column.

In the above table all wages on the American Railroads have been reduced on the basis previously stated, to equalize them with the English and Welsh prices.

All data in the above table relating to the English and Welsh Railroads have been taken from the Reports of the Board of Trade of the United Kingdom; and all relating to the American Railroads have been taken from the reports of the Railroad Commissioners of Massachusetts, embracing in all thirty-two distinct railroads.

TABLE No. 33.

Statement showing the Mileage of the various Compound Engines on the London and North Western Railway, from the Date of commencing Regular Work, to March 31st, 1885, including also the Number of Days working, and the Average Miles per Day.

Number of Engines.	Name.	Regular Working.				Number of Days, Sundays excepted, from Commencement of Regular Work, to March 31st, 1885. (313 days = 1 year.)	
		Date Commenced.	Mileage.	Working Days.			
				Number of Days.	Average Miles per Day.	Number of Days.	Average Miles per Day.
66	Experiment	April 3, 1882	174 908	616	284	937	186
300	Compound	May 2, 1883	96 132	321	299	599	160
301	Economist	May 15, 1883	93 875	312	301	588	158
302	Velocipede	May 25, 1883	89 715	316	284	579	155
303	Hydra	June 1, 1883	90 739	308	294	573	158
305	Trentham	August 8, 1883	83 512	286	292	515	162
306	Knowsley	August 16, 1883	84 104	311	270	508	166
307	Victor	August 22, 1883	85 548	313	273	503	170
310	Sarmatian	April 28, 1884	60 594	248	244	289	209
311	R. F. Roberts	March 28, 1884	53 016	188	282	315	169
315	Alaska	March 31, 1884	54 603	314	174	313	174
321	Servia	April 1, 1884	52 880	299	177	312	169
323	Britannic	April 7, 1884	44 049	254	173	307	143
333	Germanic	April 5, 1884	53 026	295	179	309	171
353	Oregon	April 13, 1884	54 590	291	177	302	181
363	Aurania	April 15, 1884	47 225	261	181	300	157
365	America	April 21, 1884	49 127	275	179	295	167
366	City of Chicago	April 23, 1884	47 243	229	206	293	161
372	Empress	October 8, 1884	25 920	106	244	149	174
374	Emperor	September 10, 1884	29 767	152	195	173	170
503	Dreadnought	September 29, 1884	17 165	59	291	157	109
504	Thunderer	March 20, 1885	1 350	8	168	9	150
508	Titan	November 7, 1884	19 484	63	309	123	158
519	Shooting Star	August 29, 1883	76 504	324	236	497	154
520	Express	August 29, 1883	86 803	314	276	497	175
1102	Cyclops	July 17, 1884	32 733	183	179	220	149
1104	Sunbeam	July 23, 1884	36 718	157	234	215	170
1111	Messenger	August 11, 1884	31 657	174	182	199	160
1113	Hecate	August 7, 1884	32 625	164	199	202	161
1115	Snake	August 14, 1884	27 818	146	190	196	142
1116	Friar	August 14, 1884	33 625	144	234	196	172
1117	Penguin	August 21, 1884	32 212	170	189	190	170
1120	Apollo	July 10, 1884	37 148	178	209	226	164
2063		May 13, 1884	36 226	218	166	276	131
	Grand Totals and Average		1 869 641	7 998	234	11 362	165

the 5:5 P. M. passenger train from Liverpool to Euston between Liverpool and Crewe, the load consisting of eighteen vehicles weighing 227 tons 15 hundred-weight, or, including the engine and tender, a total load of 292 tons 15 hundred-weight. The journey from Edge Hill to Crewe, a distance of 34¼ miles, was run in forty-five minutes, giving a speed of 43.4 miles per hour, the engine steaming freely throughout the trip."

On June 30th, 1885, the preceding mileage had been increased to 2 226 112 miles, with an average consumption of coal per mile run of 29.01 pounds, of which 1.02 pounds was consumed in starting the fires, leaving 27.99 pounds average consumption of coal per engine mile.

The preceding is a remarkably good showing, both for high average daily mileage during actual running days, and for small consumption of coal.

Mr. T. N. Ely, M. Am. Soc. C. E., General Superintendent of Motive Power of the Pennsylvania Railroad, sends the author the average locomotive mileages given in Table No. 34.

TABLE No. 34.

Statement showing Total Mileage of 48 Standard Passenger Locomotives during the Years 1882, 1883 and 1884, on Pennsylvania Railroad Division.

Mileage.	Mileage.	Mileage.	Mileage.
230 637	151 606	132 234	112 823
220 134	150 057	129 806	109 796
220 120	148 831	128 746	109 516
203 916	148 008	127 387	109 344
184 805	147 009	123 917	108 879
173 850	146 317	120 567	107 820
171 979	145 852	119 155	106 843
170 321	141 126	116 747	106 749
163 052	140 746	115 479	105 903
156 241	136 905	115 443	105 588
152 446	136 850	114 208	105 188
152 014	135 989	113 614	104 661

Total mileage of 48 locomotives, 3 years.....	6 679 224
Average " " "	139 151
" " " per year.....	46 384

Daily average (313 days = 1 year) 131 miles.

The separate mileages of these locomotive are very astonishing, averaging in three years for six days in the week as follows:

First on the above list	251	miles daily.
Second on the above list	240	"
Third on the above list	240	"
First ten on the above list	202	"

Under a recent date, Mr. Ely writes as follows to the author:

"As a matter of information bearing upon the durability of construction of American locomotives, locomotive No. 1047, class 'O,' 18 × 24, with 63" drivers, using bituminous coal, running with passenger trains on the Pittsburgh Division, ran a total of 41 510 miles in three consecutive months last. There are many other locomotives on this road that have made very large mileages, but this is the greatest for so short a period of time."

This average for ninety days of 461 miles daily, speaks well for the American engine, and also for workmanship at the Pennsylvania Railroad shop at Altoona.

The following statement has been furnished to the author by Mr. W. W. Evans, M. Am. Soc. C. E.

(Statistics given to W. W. Evans by Wm. Buchanan, Chief Engineer of Motive Power.)

Statement of Five Years' Service of Locomotives No. 10, No. 33 and No. 34, on the Hudson River Division, New York Central and Hudson River Railroad. These engines were new when they commenced running, and have been used only on through passenger trains.

Cylinders, 17" × 24"; driving wheels, 69"; fire-boxes of steel, flues of semi-steel; weight in running order, 70 500 pounds; weight on driving wheels, 44 850 pounds; average speed, 38 miles per hour; date, from September 30th, 1877, to September 30th, 1882.

Average trains consist of

2 Baggage and mail cars, loaded		52	gross tons
2 Drawing-room or sleeping cars, loaded		64	"
5 First-class passenger cars, loaded		125	"
1 Engine	31 Tons		
1 Tender, half-loaded	22 "		
		53	"
Total train		294	gross tons

TABLE No. 35.

	Locomotive No. 10.	Locomotive No. 33.	Locomotive No. 34.	Average of all three.
Total mileage in five years	415 790	440 654	392 043	416 162
Number of days in service	1 495	1 565	1 430	1 496
Number of days idle	306	242	397	315
Actual months in service	59	58	57	58
Average miles run per month	7 047.50	7 597.50	6 878	7 174.27
Average miles run each service day	278.12	281.57	274.15	277
Average miles run per year	83 158	88 131	78 409	83 233
Gallons of oil used	2 882	3 337	3 027	3 082
Average miles run per gallon	144.27	132.05	129.51	135.27
Total cost for oil	$1 182	$1 368	$1 241	$1 264
" " waste	240	232	230	234
" " labor on engine	4 066.64	4 480.24	4 343.90	4 296.93
" " " tender	285.94	447.86	380.07	371.29
" " materials E. & T.	3 218.58	4 167.39	3 621.24	3 669.07
Average cost of labor and materials used in repairs per mile run, in cents	$1\frac{82}{100}$	$2\frac{03}{100}$	$2\frac{12}{100}$	2

The following data has been kindly furnished by Mr. Walter Katte, M. Am. Soc. C. E., Chief Engineer of the New York, West Shore and Buffalo Railway.

"In relation to the comparative speed of both ordinary and extra-ordinary (or special) trains now attained and in ordinary use on first-class American railways, I beg to offer the following memoranda of what is now being done on the New York, West Shore and Buffalo Railway, which exhibits the details of an extraordinary fast run made on July 9th, 1885, by a special train between Buffalo and Weehawken, a distance of 422.6 miles.

"This train consisted of a locomotive, one baggage car and two official cars, weighing in total between 155 and 156 tons, as follows:

Make-up and weight of train, exclusive of engine—

Tender with two-third load of coal and water	62 800	pounds
Baggage car, West Shore, No. 737	46 030	"
Official car, West Shore, No. 90	61 200	"
Official car, Baltimore and Ohio, No. 711	46 430	"
Total	216 460	"

Add for total weight of train—

On Buffalo Division, West End, Engine No. 45, Class B (bituminous), 94 500 pounds. Total	310 960	pounds
Equal to about	155	tons

On Buffalo Division, East End, Engine No. 50, Class B, 94 500 pounds. Total	310 960 pounds
Equal to about	155 tons
On Hudson River Division, West End, Engine No 27, Class A (anthracite), 96 000 pounds. Total	312 460 pounds
Equal to about	156 tons
On Hudson River Division, East End, Engine No. 36, Class B, 94 500 pounds. Total	310 960 pounds
Equal to about	155 tons

"The train (see Table 36) started from East Buffalo at 10:04 A.M., and reached Weehawken at 7:27 P.M., or 9 hours 23 minutes on the trip, making 19 stops consuming 2 hours, reducing the actual running time to 7 hours 23 minutes, or a general average of about 56 miles per hour running time. This train attained a speed of 87 miles per hour between Churchville and Genesee Junction, on the west end of the Buffalo Division; and at several other places attained speeds from 70 to 80 miles per hour.

"It is interesting to state the fact that this train was run without any preconcerted effort or preparation for it. It was run by four different engines, which were called on to take this 'special' just as they stood in their respective yards waiting to take their 'regular' trains. Nor were the enginemen informed beforehand that they were expected to make more than ordinary speed; they were simply told to see how fast they could run after getting started. The train was run subject to all the exigencies and privileges of the regular trains on the line, none of which were obstructed or interfered with. The several engines, when through with their runs, did not show the slightest signs of having made any extra effort, and were in perfect condition to turn right round and make their runs back again, if it had been desired; neither was the steam pressure greater, nor the amount of fuel consumed greater than allowable or out of proportion to the speed attained.

"Table No. 37 gives the time of nineteen regular passenger trains, running on the present summer schedule, between Weehawken and Buffalo, Syracuse, Schenectady, Albany and Kingston, which shows their average speeds to be from 28 to 37 miles per hour, including stops, and from 33 to 50 miles per hour excluding stops—on regular schedule time. When making up 'lost time,' it is not uncommon for these trains to be run at speeds fully up to 60 miles per hour between stations.

TABLE No. 36.

Summarized Record of Fast Run on the New York, West Shore and Buffalo Railway on July 9, 1885, from East Buffalo to Weehawken, 422.6 Miles. Principal Stations only.

Running Record.				Speed Record.†											
Distance Miles.	Stations.	Time.	Remarks.*	East Buffalo.	Genesee Junction.	Newark.	Syracuse.	Utica.	Frankfort.	Canajoharie.	Rotterdam Junction.	Coeymans.	Kingston.	Cornwall.	Haverstraw.	Weehawken.
0.0	Buffalo															
3.4	East Buffalo	10:04	Took train from Niagara Falls Branch	....	68	67	64	62	62	60	60	60	58	58	57	57
64.3	Genesee Junction	11:00 11:07	B., N. Y. & P. grade crossing. Took water and oiled	....	....	65	61	59	60	58	58	59	56	56	55	56
96.8	Newark	11:41 11:50	Changed engines	....	....	....	59	57	59	57	57	58	55	55	54	55
147.7	Syracuse	12:48 12:55	Stopped for lunch and took water	....	....	....	....	56	58	56	57	57	55	54	54	55
194.2	Utica	1:54		....	....	....	....	....	73	55	57	58	54	54	53	54
205.1	Frankfort	2:04 2:10	Changed engines	....	....	....	....	....	....	51	55	57	53	53	52	54
235.7	Canajoharie	2:48 2:56	Stopped for water	....	....	....	....	....	....	....	60	60	54	54	53	54
266.5	Rotterdam Junction	3:29 3:29	Stopped by block	....	....	....	....	....	....	....	....	61	52	51	51	53
297.8	Coeymans	4:07 4:13	Changed engines	....	....	....	....	....	....	....	....	....	46	50	49	52
337.8	Kingston	5:01 5:01	Grade crossing	....	....	....	....	....	....	....	....	....	....	54	51	55
373.7	Cornwall	6:20		....	....	....	....	....	....	....	....	....	....	....	46	55
392.8	Haverstraw	6:50 6:51	Stopped by block	....	....	....	....	....	....	....	....	....	....	....	....	62
426.0	Weehawken	7:27														

* In addition to the stops noted in this column, the following were made: Red Creek, 11:11 to 11:11, grade crossing; Lyons, held 2 minutes by freight; Clark's Mills, 1:41 to 1:41, grade crossing; So. Schenectady, 3:39 to 3:40, grade crossing; Voorheesville, 3:52 to 3:52, grade crossing; Glenerie, stopped 3 minutes for water; Marlborough, 5:30 to 5:39, stopped by block; Newburgh, 5:59 to 6:00, stopped by block; New Durham, 7:23 to 7:23, stopped by block.

† Speed is given in nearest even miles per hour deducting actual stops, plus two minutes per stop for losing and gaining headway.

TABLE No. 37.

Speed Table New York, West Shore and Buffalo Railway.—Time Table No. 11, July 2d, 1885.

Train No.	Miles Run.	Between Weehawken and	Time.			Stops.			Actual Time Running.	Average Miles Per Hour.	
			Leave.	Arrive.	Consumed.	Station.	Junction.	Time Lost.		Including Stops.	Excluding Stops.
					H. M.			H. M.	H. M.		
59	422.6	Buffalo	7:40	9:50	14 10	93	6	5 40	8 30	29.83	49.71
63	152.6	South Schenectady	9:20	1:30	4 10	4	1	25	3 45	36.62	40.69
51	422.6	Buffalo	10:30	11:55	13 25	16	7	2 03	11 22	31.45	37.17
69	88.2	Kingston	11:35	2:10	2 35	5	0	16	2 19	34.14	38.07
53	278.3	Syracuse	11:45	8:55	9 10	46	5	2 57	6 13	30.36	44.76
67	152.6	South Schenectady	4:05	8:10	4 05	4	1	25	3 40	37.37	41.61
65	141.3	Albany	4:15	9:05	4 50	26	1	1 32	3 18	29.23	42.81
57	422.6	Buffalo	6:30	6:00	11 30	8	7	1 19	10 11	36.74	41.49
55	422.6	"	8:30	11:40	15 10	19	7	2 18	12 52	27.86	32.84
62	422.6	"	3:40	6:30	14 50	46	6	3 22	11 28	28.48	36.83
54	422.6	"	4:50	7:00	14 10	11	7	1 23	12 47	29.83	33.05
66	141.3	Albany	4:10	9:05	4 55	27	1	1 35	3 20	28.73	42.39
52	422.6	Buffalo	9:20	10:20	13 00	16	7	1 53	11 17	32.50	37.45
56	278.3	Syracuse	5:00	2:40	9 40	50	3	3 08	6 32	28.78	42.59
64	152.6	South Schenectady	1:35	6:00	4 25	6	1	32	3 53	34.55	39.29
60	422.6	Buffalo	5:10	7:00	13 50	68	6	4 27	9 23	30.54	45.03
70	88.2	Kingston	5:35	8:00	2 25	4	0	13	2 12	36.49	40.09
68	152.6	South Schenectady	5:13	9:20	4 7	5	1	28	3 39	37.06	41.80
58	422.6	Buffalo	9:30	9:50	12 20	14	6	1 53	10 27	34.26	40.44

"The character of the engines doing this service, with data of their dimensions, weights, etc., is as follows:

General dimensions, weights, etc., of locomotives. Class A (anthracite) and B (bituminous):

Cylinders—	
Diameter and stroke	18 x 24 inches
Ports—length	16 "
Steam port—width	1½ "
Exhaust port—width	3¼ "
Valves: Allen Richardson—	
Travel (maximum)	5½ "
Outside lap	1 "
Inside lap	1/16 "
Lead in full gear	1/32 "
Exhaust nozzles: high double—	
Diameter	3⅛ "
Height from base of stack	18 "
Boiler pressure per square inch	140 pounds
Smoke-stack—Height above rail	14 ft. 5¼ inches
Diameter at top	20½ "
Diameter at base	18 "
Smallest internal diameter	15 "
Engine and tender truck wheels: Allen paper.	
Capacity of tender—	
Coal	15,000 pounds
Water	3,000 gallons
Boiler—	
Diameter of smallest ring	55 inches
Length of barrel	10 ft. 11 "
Tubes—	
Number	188
External diameter	2 inches
Length between sheets	10 ft. 10⅝ "
Area through	3.17 square feet

Heating surface--

Tubes	1,084 square feet
Fire-box	128 "
Total	1,212 "

Firebox--

	Class A.	Class B.
Length outside	10 ft., 9 inches	6 ft., 7 inches
Width outside	48 "	$41\frac{3}{4}$ "
Depth inside front	$49\frac{3}{4}$ "	$74\frac{3}{4}$ "
Size of grate	$120\frac{3}{8}$ x $40\frac{1}{2}$ "	$70\frac{3}{8}$ x $34\frac{1}{2}$ "
Grate area	34 square feet	17 square feet
Weight of engine in working order	96,000 pounds	94,500 pounds
" " on truck	32,000 "	32,000 "
" " on drivers	64,000 "	62,500 "
" tender loaded	76,000 "	76,000 "
" " empty	36,400 "	36,400 "
Maximum weight, engine and tender	172,000 "	170,500 "
Average " " " "	154,000 "	154,000 "

"Their trains vary from four to twelve cars, and often have on one train three baggage and express cars, and three or four Pullman parlor and sleeping cars.

TABLE No. 38.

Physical Characteristics of West Shore Road.

Elements.		Buffalo Division.		Hudson River Division.		Entire Line.
		West End.	East End.	West End.	East End.	
Profile.						
Level	Percentages.	16.93	27.03	29.94	42.18	30.18
Up grade (going East)	Percentages.	45.59	37.50	29.59	30.41	35.62
Down " " "	Percentages.	37.48	35.47	40.47	27.41	34.20
Up grade (going East)	Average feet per mile.	16.25	15.88	16.36	16.96	16.32
Down " " "	Average feet per mile.	18.75	18.49	17.66	22.77	19.51
Alignment.						
Tangents	Percentages	82.52	83.44	73.85	84.26	82.06
Curves	Percentages	17.48	16.56	26.15	15.74	17.94
Curvature, average, degrees		1° 20' 57"	1° 46' 11"	2° 04' 40"	1° 48' 17"	1° 46' 34"

"It will be seen by inspecting the above table of the physical characteristics of the road, that it is unusually (for the Eastern States) favorable for making fast time with small effort, its grades over the entire line averaging only $16\frac{1}{3}$ feet per mile rising east, and 19.5 feet per mile going west, while the percentage of grade to level line is 70 per cent. of gradients to 30 per cent. of level—the upward and downward gradients being about equal, or 35 per cent. each; so that a train running in either direction always has the advantage of 30 per cent. of level and 35 per cent. of down grades in its favor.

"The percentage of straight line to curves is also remarkably high, being 82 per cent. straight to 18 per cent. curve. The average degree of curvature over the entire line is about $1\frac{3}{4}$ degrees, the standard maximum permanent curvature being 4 degrees.

"The road bed and bridges are of thoroughly first-class character; the track is double and laid on the break-joint-system, with 67 pound steel rails on cross-ties averaging 2 800 to the mile; the joints are made with rolled-iron double 'angle splice' bars, weighing 44 pounds per pair, and are 36 inches in length, with three cross-ties under the joints covering the whole length of the splice bars; the track is well ballasted throughout, generally with good gravel, with a considerable proportion of machine-crushed stone ballast."

Endurance of Locomotives.

In order to compare the endurance of the locomotives of the two countries, Table No. 39 was constructed. For the English railroads the data were taken from the Board of Trade Reports; for the American, from the Reports of the Railroad Commissioners of Massachusetts, Reports of the Companies and Poor's Manual.

For the English railroads the average mileage is very uniform, being from 60 to 70 miles daily for 313 days in the year.

The American railroads do not show such uniform results, the mileage being on the smaller Massachusetts railroads about 60 miles; on the Pennsylvania and New York Central, 80 miles; and on the New York, New Haven and Hartford, 96 miles. The last is a remarkable record of locomotive endurance or durability, especially as it was the same for the two previous years.

TABLE No. 39.

Showing the Average Annual Revenue Mileage made by all the Locomotives in 1884 on all the Principal Railroads of the United Kingdom and some of the Principal Railroads of the United States.

Name of Railroad.	Total Number of Locomotives.	Mileage.	
		Total.	Average.
United Kingdom.			
Great Eastern	655	14 053 700	21 458
Great Northern	789	16 978 030	21 498
Great Western	1 577	30 457 257	19 314
London and North Western	2 476	38 184 328	15 422
London and South Western	501	11 183 874	22 323
London, Brighton and South Coast	410	8 136 077	19 844
London, Chatham and Dover	180	3 889 988	21 611
Midland	1 697	33 261 729	19 600
North Eastern	1 470	23 873 903	16 241
Caledonian	690	12 489 848	18 101
North British	585	11 676 275	19 958
Great Northern of Ireland	127	2 636 570	20 760
Great Southern and Western of Ireland	163	2 913 669	17 874
Totals and Average	11 320	209 735 248	18 528
United States.			
Pennsylvania:			
Pennsylvania Railroad Division	815	20 791 778	25 511
United Railroads of New Jersey Division	335	8 408 926	25 101
Philadelphia and Erie Division	110	2 353 131	21 392
Baltimore and Ohio	603	11 405 777	18 915
New York Central and Hudson River	657	16 452 835	25 043
New York, Lake Erie and Western	805	17 092 582	21 233
Boston and Albany	243	4 960 185	20 407
Fitchburg	100	1 986 319	19 863
Boston and Lowell	147	1 915 312	13 028
Boston and Maine	92	1 829 573	19 887
Eastern	115	2 142 225	18 628
Boston and Providence	57	973 646	17 081
New York and New England	147	2 350 058	15 986
Old Colony	125	2 428 954	19 432
New York, New Haven and Hartford	128	3 827 685	29 904
Totals and Average	4 479	98 918 986	22 085

Locomotive Earnings.

In order to ascertain the money-earning qualities of the locomotives of each country, Table No. 40 was constructed, in which fourteen railroads, embracing all the principal roads of the United Kingdom, are compared with an equal number of American roads.

TABLE No. 40.

Showing the Average Annual Earnings of Locomotives on Different Railroads in 1883.

Name of Railroad.	Total Number Locomotives.	Earnings from Traffic.	
		Total.	Average per Locomotive.
United Kingdom.			
Great Eastern	615	£3 279 354	£5 330
Great Northern	733	3 522 035	4 805
Great Western	1 577	7 817 742	4 957
London and North Western	2 451	10 362 520	4 228
London and South Western	471	2 797 550	5 940
London, Brighton and South Coast	410	2 114 879	5 158
London, Chatham and Dover	168	1 095 317	6 520
Midland	1 629	7 363 217	4 520
North Eastern	1 462	6 699 891	4 583
South Eastern	325	1 949 391	6 000
Caledonian	690	2 923 890	4 237
North British	573	2 611 803	4 558
Glasgow and South Western	290	1 111 796	3 834
Great Southern and Western of Ireland	160	753 396	4 709
Totals and Average	11 554	£54 402 781	£4 709
United States.			
Pennsylvania—Pennsylvania Division	795	$32 017 813	$40 273
Do. United Railroads of New Jersey	330	14 408 540	43 662
Do. Philadelphia and Erie	110	4 108 843	37 353
New York, Lake Erie and Western	785	22 040 595	28 077
New York Central and Hudson River	655	29 359 745	44 824
Boston and Albany	244	8 103 957	33 213
Fitchburg	98	2 825 024	28 827
Boston and Lowell	77	2 102 887	27 310
Boston and Maine	89	2 915 944	32 763
Eastern	109	3 417 267	31 351
Boston and Providence	53	1 646 962	31 075
New York and New England	151	3 376 311	22 360
Old Colony	120	4 158 977	34 658
New York, New Haven and Hartford	122	6 650 549	54 513
Totals and Average	3 738	$137 133 414	$36 686

Notwithstanding that the average charges, both for passengers and freight, are much lower in the United States than in the United Kingdom, the average earnings of the locomotives on the above fourteen American railroads are 55 per cent. greater than the average of the fourteen roads of the latter country as named above. This speaks well for the money-making quality of the American locomotive, as well as for its durability.

The London and North Western Railway has about the same amount of traffic-earnings as the three divisions of the Pennsylvania Railroad

named (the other divisions have been excluded, as being in the level Western States), yet the latter company only has about half the number of locomotives, notwithstanding this road has very heavy grades, and with one summit 2 154 feet above the sea.

Relation between Train Load and Charges.

Table No. 41, compiled from the Reports of the New York Central and Hudson River Railroad, shows the cost of transporting a ton of freight for 14 years—from 1870 to 1883, inclusive—and also the average load of freight trains for each year. It will be noticed that the average earnings or charges per ton per mile have decreased one-half, and the average loads of freight trains have doubled within the 14 years.

TABLE No. 41.

Freight Earnings, Expenses and Profit per Ton-mile, and per Train-mile, and average Train-load.

Year Ending September 30th.	Earnings per ton-mile on freight.	Expenses per ton-mile on freight.	Profit per ton-mile on freight.	Earnings per train-mile on freight.	Expenses per train-mile on freight.	Profit per train-mile on freight.	Average number of tons of freight in train-load.
	Cents.	Cents.	Cents.				
1870	1.88	1.15	0.73	$1.95	$1.19	$0.76	103
1871	1.62	1.01	0.61	2.07	1.28	0.79	127
1872	1.59	1.12	0.47	2.05	1.45	0.60	129
1873	1.57	1.02	0.55	2.02	1.32	0.70	129
1874	1.46	0.98	0.48	2.03	1.37	0.66	139
1875	1.27	0.90	0.37	2.11	1.49	0.62	166
1876	1.05	0.71	0.34	1.89	1.28	0.61	180
1877	1.01	0.69	0.32	1.68	1.15	0.53	166
1878	0.93	0.59	0.34	1.71	1.10	0.61	183
1879	0.78	0.54	0.24	1.52	1.05	0.47	191
1880	0.87	0.54	0.33	1.92	1.18	0.74	218
1881	0.78	0.56	0.22	1.70	1.23	0.47	217
1882	0.73	0.60	0.13	1.62	1.31	0.31	219
1883	0.91	0.68	0.23	1.82	1.37	0.45	199

A similarly constructed table for the Pennsylvania Railroad shows that, within the last 20 years, the average freight charges have been reduced two-thirds, while the average load of freight trains has been more than doubled.

The workings of other American railroads also show the great economy in running heavy freight trains.

The English roads, on account of their good alignment and easy

grades, ought to be able to run heavier freight trains than the American, with proportionate saving in cost; this would enable them to reduce their charges, or pay larger dividends.

Other causes have contributed to this reduction in the cost of transportation, besides the increased load of freight trains; this, however, must have great influence on the result, as now one engine and one set of men do more work than two engines and two sets of men did formerly.

The author has been desirous to compare impartially and closely the operating expenses of the railroads of the United Kingdom and those of the United States—the result of comparing the cost of motive power on the two systems has been as great a surprise to him as it will be to any one.

At the very outset he was met with the apparently insurmountable obstacle that the English do not keep any accounts showing the passenger or ton mileage, which is so necessary for comparing operating expenses. After diligent investigation and research, he made his rules for estimating and supplying this deficiency (see page 16). Of course these are only his views and analysis of the data available to him, which he is willing to change promptly by any reliable or official data. He hopes that the railroads which are so largely interested in this question, will promptly replace his estimates by their official figures. Until this is done, he claims that these figures should be accepted as correct.

In conclusion, the author begs the indulgence of his readers for the imperfection of this paper. The data was collected, compiled and analyzed while he was very closely occupied in other professional duties. He is aware that the arrangement of many of the tables could be much improved and others advantageously made, but he cannot do this, for the want of the necessary time. Doubtless, many clerical errors will be discovered, as for want of time many calculations have not been checked.

All who are interested in this line of study can, from the data here given, make their own tables and draw their own conclusions, as the author has done.

Conclusions.

In the opinion of the author, neither the English or American railroad is perfect in itself, if the object of a railroad is to give the greatest amount of comfort to the passengers for the least amount of money, and the cheapest freight charge to the shipper. In this light the Pennsylvania Railroad comes nearer perfection than any other, but it has the serious fault of many level road-crossings. All other large American railroads have, in addition to this defect, that of the absence of the block system. With these two faults alone, they are far from being perfect, to say nothing of minor improvements required, such as good station buildings, etc.

Although the English cars are far from being as comfortable for travelers as the American, yet the English engineer can do but little now to remedy it, owing to the low and narrow cars he is obliged to run. Perhaps the most that could be done would be economising in the cost of motive power, the general introduction and use of the American baggage check on all connecting lines, thereby increasing the comfort of passengers, and also saving a large expense to the railroads in saving the cost of many unnecessary porters; warming the cars in winter; general use of the bogie truck under passenger cars, thereby diminishing the constant jarring motion now so great in them.

The English road-bed, superstructure and block signaling system is all that could be desired.

From superficial observation, it is difficult to say what is the cause of the great additional cost in motive power in operating the English railroads, when it should be much less than that of the American railroads, as, owing to their average superior construction, with easier grades and curves, it should not be much over half, instead of being nearly double.

Apparently the most prominent cause of this increased cost is the great speed and small tonnage of the freight trains, and too many passenger trains lightly loaded.

Perhaps the American bogie-truck rolling stock runs with less friction than the rigid wheel-base rolling stock used on the English roads.

The English railroads have cost per mile more than three times as much as the American, yet they only save eight per cent. of the annual operating expenses. In other words, to save one-twelfth of the annual

operating expenses the cost of construction has been increased more than three times. The preceding comparison would be still more striking if we brought into calculation the comparative cost in the two countries of the different items included under the heading of "Maintenance of way," "Locomotive Charges" and "Repairs of Carriages." Probably 90 per cent. of these are made up of labor, fuel, iron, steel, etc., all of which would average, in 1883, fully one-fourth higher in the United States than in the United Kingdom. This speaks very well for the American system, that, notwithstanding it costs much less to construct, after paying higher prices for labor and materials—and for financial reasons the location is often faulty, and the superstructure of perishable materials—yet it is operated for less than the English roads are, when due allowance is made for the difference in the price of labor and materials that enter into operating expenses. Notwithstanding these disadvantages, for similar accommodation, the passenger rates and freight charges are much less on American than on English railroads.

One of the principal items of the greater cost of English railroad construction over the American, is the necessity of having much straighter alignment or easier curves, so that it can be safely operated by the rigid and long wheel-base rolling stock in use there.

The Baltimore and Ohio Railroad is a sample of what can be done with the American rolling stock. This road is built through a very difficult and rugged country, which compelled a very poor alignment, with nearly one-half of the entire length in curvature, which curves run up to 600 feet radii, and long grades running up to 120 feet per mile. The country affords no natural advantages whatever. Yet, with all these drawbacks, this road does a very large and profitable business, paying annually ten per cent. dividend, and running passenger trains safely at very high speed.

All this is done on a road that could not be operated with rolling stock built on the English system. The extra cost of enlarging these curves to adapt them to English rolling stock would be so great as to be commercially impracticable. It is not difficult to appreciate the great difference in cost of construction, in an extremely rough country, of a railroad with curves 600 feet, or 2 640 feet radii.

Unquestionably the American system of construction is the best for new countries, or where cheapness of construction is desirable. The American rolling stock, with the bogie truck, will run safely and rapidly

over roads of inferior construction, or sharp curves that would be impossible for rolling stock constructed on the English type of long and rigid wheel base. The American type is especially adapted for military purposes. During the late American war some military railroads were operated successfully, with the ordinary American rolling stock, with curves of 50 feet radius. The New York Elevated Railroad has been operated for years (it transported last year 97 000 000 passengers) without accident, and has many curves under 100 feet radius. Through an ordinary rough country, a railroad to be operated with the American type of rolling stock could be constructed in one-fourth of the time and for one-fourth of the money that one suitable for the English rolling stock could be built.

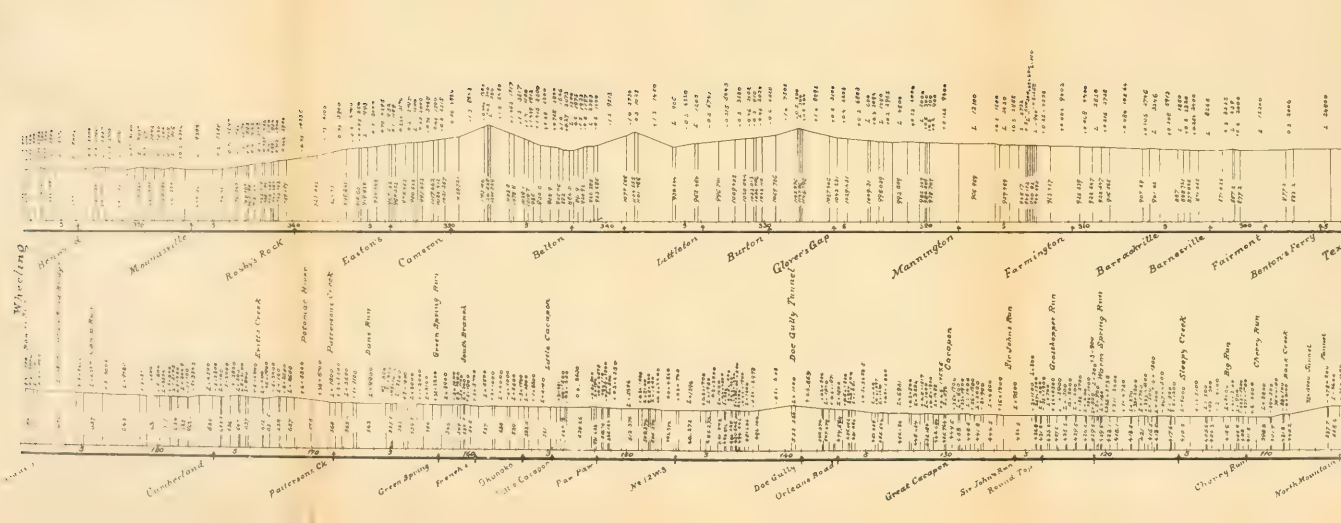
It would certainly pay the management of the English railroad companies to investigate the cause of the extra cost of motive power on their roads, and, if possible, remedy it. If this can be done, they will be able to *decrease their operating expenses* over 8 per cent., without making any changes whatever in the present prices. This will enable most companies to *increase their dividends* largely—probably over 4 per cent.

For what *is* done in the United States *ought* to be done in the United Kingdom.

PROFILE OF THE PALT'MORE AND OHIO RAILROAD

FROM

BALTIMORE TO WHEELING



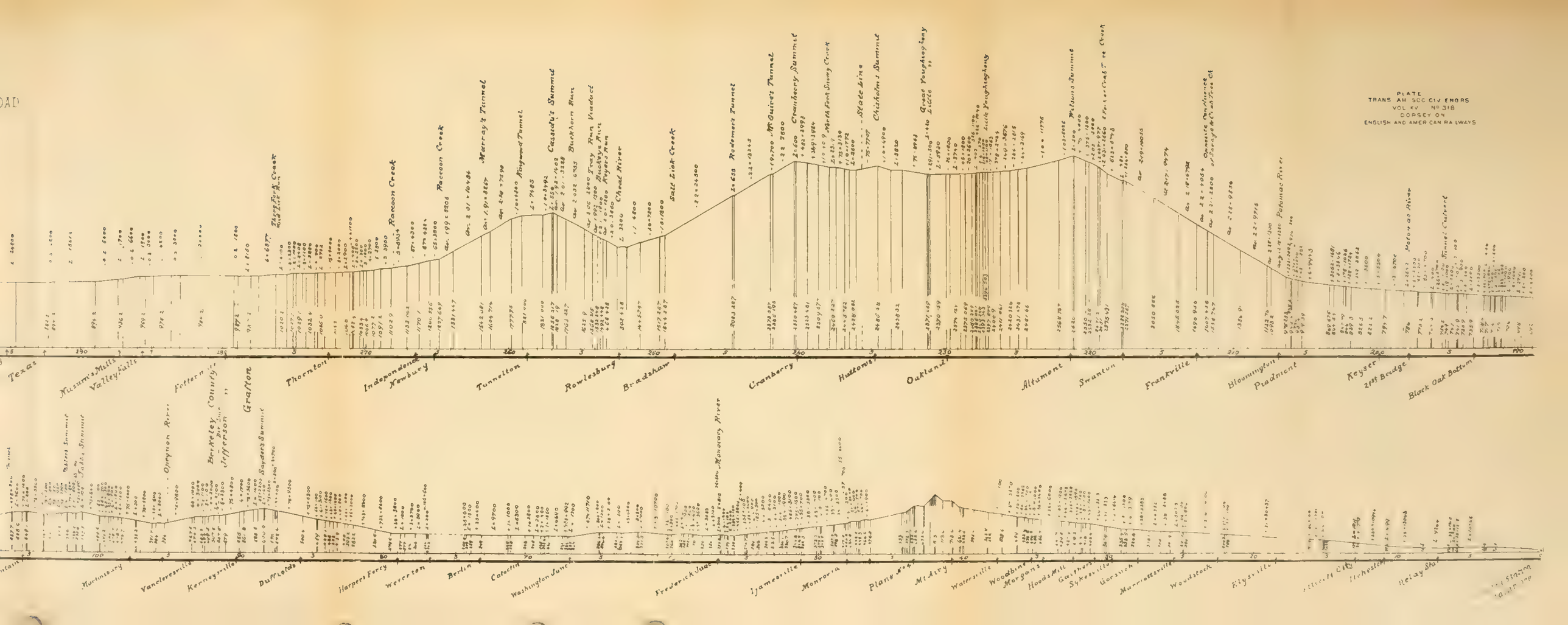
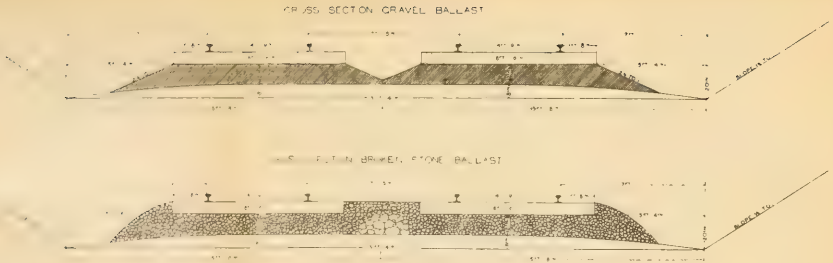
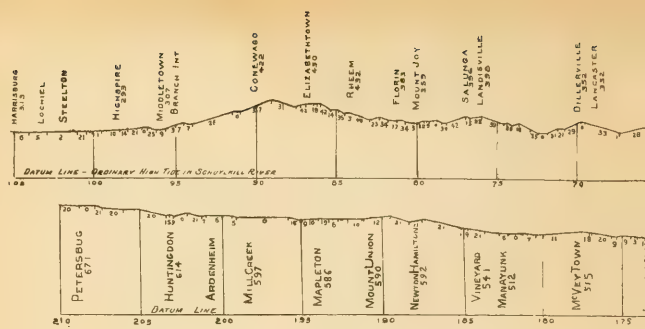
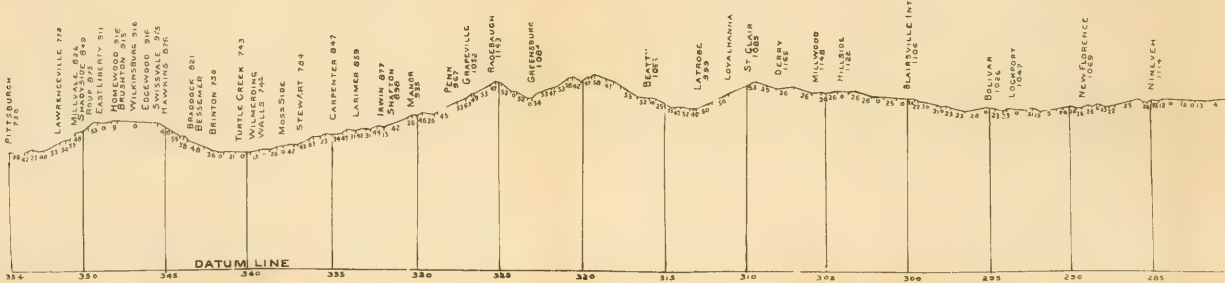


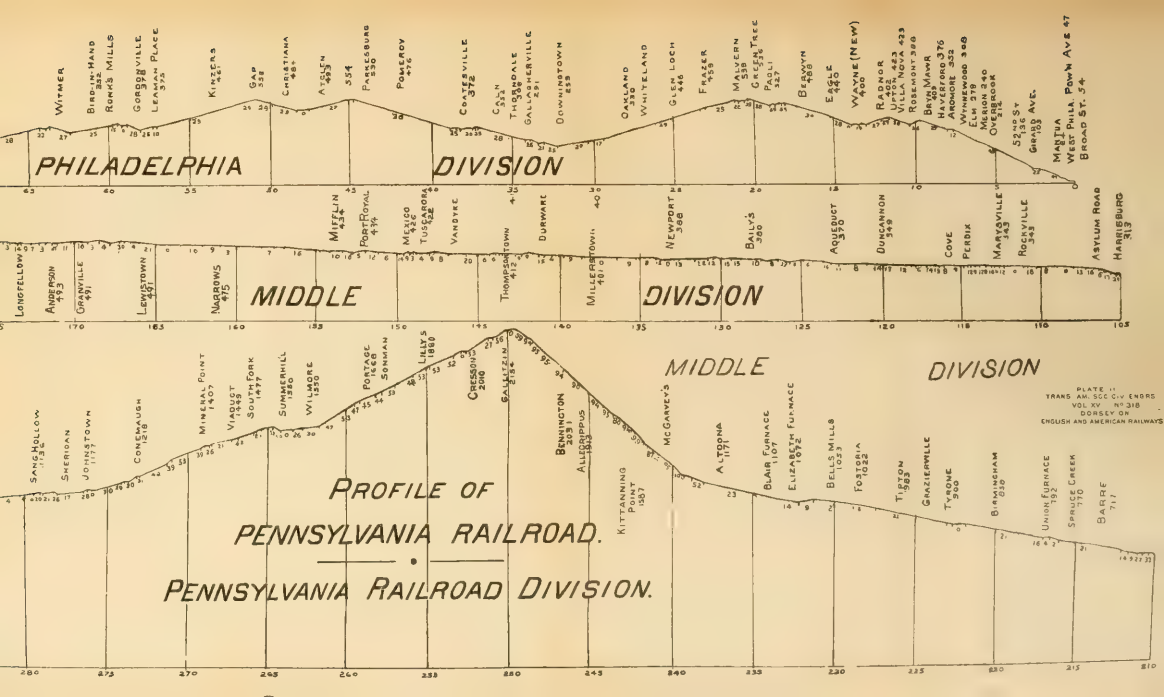
PLATE
TRANS AM SOC CIV ENGRS
VOL LV NO 318
DORSEY ON
ENGLISH AND AMERICAN RAILWAYS
Texas
Nuzum's Mills
Valley Falls
Fetterman
Thornton
Independence
Newbury
Tunnelton
Rowlesburg
Bradshaw
Cranberry
Hutton's
Oakland
Altamont
Swanton
Frankville
Bloomington
Piedmont
Keyser
21st Bridge
Black Oak Bottom
Grafton
Berkeley County
Jefferson
Opequon River
Martinsburg
Vanclevesville
Kennesville
Duffields
Harpers Ferry
Weverton
Berlin
Catoctin
Washington Junc
Frederick Junc
Ijamesville
Monrovia
Plane No 4
Mt Airy
Waterville
Woodbine
Morgans
Hoods Mill
Gaithers
Sykesville
Gorsuch
Marriottsville
Woodstock
Elysville
Relay Sta





PITTSBURGH DIVISION




PHILADELPHIA DIVISION
MIDDLE DIVISION
MIDDLE DIVISION
PROFILE OF
PENNSYLVANIA RAILROAD.
PENNSYLVANIA RAILROAD DIVISION.
PLATE II
TRANS. AM. SOC. CIV. ENGRS
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ENGLISH AND AMERICAN RAILWAYS

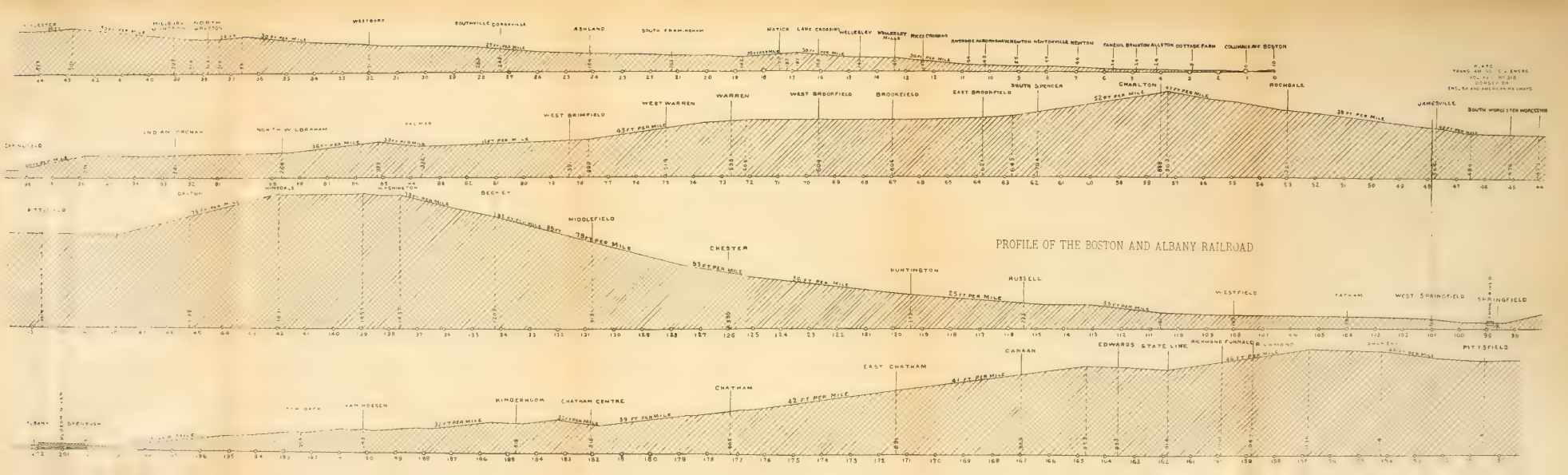
PROFILE OF THE BOSTON AND ALBANY RAILROAD
BOSTON
COLUMBIA AVE
COTTAGE FARM
ALLSTON
BRIGHTON
NEWTONVILLE
NEWTON
WELLESLEY
WELLESLEY HILLS
NATICK
SOUTH FRAMINGHAM
ASHLAND
SOUTHVILLE
WESTBORO
HILLSIDE
NORTH WINTHROP
WINTHROP
ROCHDALE
CHARLTON
SOUTH SPENCER
EAST BROOKFIELD
BROOKFIELD
WEST BROOKFIELD
WARREN
WEST WARREN
WEST BRIMFIELD
PALMER
NORTH W. ABRAHAM
INDIAN ORCHARD
SPRINGFIELD
WEST SPRINGFIELD
WESTFIELD
RUSSELL
HUNTINGTON
CHESTER
MIDDLEFIELD
PITTSFIELD
CANAAN
EAST CHATHAM
CHATHAM
CHATHAM CENTRE
KINDERHOOK
VAN HOESEN
ALBANY
RENSSELAER
EDWARDS
STATE LINE
RICHMOND FURNACE
RICHMOND
SHUNTS
JAMESVILLE
SOUTH WORCESTER
WORCESTER

TABLE OF DIVISIONS

DIVISION No 1			
MAIN LINE	BOSTON TO WORCESTER	44.10 MILES	DOUBLE TRACK
GRAND JUNCTION BRANCH	COTTAGE FARM TO EAST BOSTON	3.10 "	" "
NEWTON HIGHLANDS BRANCH	BROOKLINE JUNCTION TO COOK STREET	0.85 "	" "
NEWTON LOWER FALLS BRANCH	RIVERSIDE TO NEWTON LOWER FALLS	1.22 "	SINGLE TRACK
SAXONVILLE BRANCH	NATICK TO SAXONVILLE	3.70 "	" "
MILFORD BRANCH	SOUTH FRAMINGHAM TO MILFORD	2.00 "	" "
MILLBURY BRANCH	MILLBURY JUNCTION TO MILLBURY	3.00 "	" "
DIVISION No. 2			
MAIN LINE	WORCESTER TO SPRINGFIELD	54.53 MILES	DOUBLE TRACK
WEBSTER BRANCH	WEBSTER JUNCTION TO WEBSTER MILLS	1.01 "	SINGLE TRACK
SPENCER BRANCH	SOUTH SPENCER TO SPENCER	1.18 "	" "
NORTH BROOKFIELD BRANCH	EAST BROOKFIELD TO NORTH BROOKFIELD	4.16 "	" "
WARE RIVER BRANCH	PALMER TO WINCHENDON	49.20 "	" "
ATHOL BRANCH	ATHOL JUNCTION TO ATHOL	45.26 "	" "
DIVISION No. 3			
MAIN LINE	SPRINGFIELD TO PITTSFIELD	51.93 MILES	DOUBLE TRACK
NORTH ADAMS BRANCH	NORTH ADAMS JUNCTION TO NO. ADAMS	18.00 "	SINGLE TRACK
DIVISION No. 4			
MAIN LINE	PITTSFIELD TO ALBANY	51.54 MILES	DOUBLE TRACK
HUDSON BRANCH	CHATHAM TO HUDSON	17.17 "	SINGLE TRACK

PLATE IV
TRANS AM SOC CIV ENGRS
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DORSEY ON
ENGLISH AND AMERICAN RAILWAYS

SUMMARY

DIV.	DOUBLE TRACK	SINGLE TRACK	TOTAL
No 1	60.05 MILES	19.99 MILES	78.04 MILES
No 2	54.53 "	11.09 "	65.44 "
No 3	51.93 "	18.00 "	69.93 "
No 4	51.54 "	17.17 "	68.71 "
TOTAL	218.05 "	166.07 "	382.12 "

MAIN LINE CAPACITY TABLE

Guage 4 ft 8½ inches		Space between tracks 6 ft	
DIVISION	CLEAR HEIGHT	MAXIMUM DEGREE	CURVATURE RADIUS
No 1	14 FT 5 INCHES	10 DEGREE	573 FEET
No 2	17 0 "	5 "	1146 "
No 3	17 6 "	6 "	955 "
No 4	16 3 "	7 "	818 "

TOTAL LENGTH OF CURVED LINE, 112.29 MILES
TOTAL NUMBER DEGREES TURNED, 8225

SPRINGFIELD, 1885

SHEET No 1045

ROAD DEPARTMENT

BOSTON AND ALBANY R.R.

PROFILE SCALES
(SEE PLATE III)
HORIZONTAL, ½ INCH = 1 MILE
VERTICAL, 1 INCH = 600 FT.

W. H. RUSSELL CHF. ENGINEER.
G. B. Hardy ASSIST "

E. Alger DIV ENGINEER.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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319.

(Vol. XV.—February, 1886.)

THE DISINTEGRATION OF THE EGYPTIAN OBELISK IN THE CENTRAL PARK, NEW YORK.

By THOMAS EGLESTON, PH.D., M. Am. Soc. C. E.

READ JANUARY 20TH, 1886.

The decay of the stone of the Obelisk in the Central Park of New York has attracted a great deal of attention, on account of the publicity which has been given to its having been coated with paraffine in order to preserve it. I have therefore thought it would be of interest to this Society to know of the results of a special examination which I have made of it, in addition to those already communicated.* It was supposed when the Obelisk was brought here that it was in a perfect condition, and that it had not suffered at all by its exposure to the climate of Egypt for 3 500 years. A careful examination of the Obelisk itself, and of the pieces which have been detached from it, shows, however, that this was not the case, but that disintegration has been going on for a very long time in the interior of the stone, which has only become apparent within a few months, owing to the fact that, until its erection in the Central Park, there were no causes which

* Decay and Preservation of Building Stone. By T. Egleston, Ph. D. Annual Convention Am. Soc. Civil Engineers, June, 1885. To be published hereafter.

tended to attract attention to the weakness of the stone. For some months it has been noticed that pieces of the Obelisk have been falling about the base. These pieces, mostly of small size, attracted the attention of those interested in its preservation, and led Mr. A. Hague, during the last summer, to make an investigation of the condition of the whole shaft while the scaffolding used by the workmen in putting on the paraffine coating was still up and they were chipping off the cracked pieces and heating the surface of the obelisk preparatory to putting on the paraffine. This examination showed that the south and west faces, which were exposed to the direct rays of the sun, were the most weathered, but that it was cracked on all sides, showing that a process of decay had begun which is likely to lead to most serious consequences if the Obelisk is not protected.

The process of decay through which the stone is passing is not of the chemical nature which has been attributed to it. This is shown not only by an inspection of the Obelisk itself, but by the microscopic examination of the pieces of the stone detached from it. The first thing done by the workmen in getting the stone ready for the coating which was to protect it, was to pick off all the pieces which were already so much cracked as to be easily separated, with small iron tools. In doing this, pieces of an inch square up to those which were over a foot long and nearly two inches in thickness were detached. These pieces appear to the unaided eye to be of perfectly natural stone. Many of them are coated with a green substance which I at first sight supposed to be a mineral, and which, from its peculiar color, I took to be epidote. A careful microscopic examination, however, did not show any crystals or fibers, which made me withdraw the determination, and decide that it must be some kind of vegetable stain. Mr. P. H. Dudley, who has made a careful microscopic examination of this stain, pronounces it to be a very low order of vegetable growth belonging to the genus *Protococcus*, probably the species *pluvialis*, a variety not found in this country, and known mostly in Egypt. What strikes one at first, in making an examination of the pieces of stone, is their remarkably fresh appearance, except where they are colored green. The crystals of feldspar are large and rosy, and do not, to the eye, show any sign or trace of decomposition. Where the fracture of the stone is quite fresh it looks like any other syenitic granite, with a considerable preponderance of hornblende, and but a small quantity of mica. To the naked eye, only quartz, feldspar, hornblende and mica

are visible. Under the microscope, tourmaline, zircon, titanite and magnetite are seen in small quantities with perfectly fresh fractures, and not showing the least sign of decomposition. With a 1½-inch objective and powerful light, I first began to notice small cavities, here and there, of irregular shape, and quite deep, sometimes entirely empty, but often containing a substance having very much the appearance of the hornblende and pyroxene in specimens which occur in limestone, and have been placed in acid to detach the crystals. These cavities, though not numerous, occur only in the vicinity of the hornblende. On searching for them I found them relatively numerous when all the surfaces of attack were taken into consideration, and sometimes so deep that their bottom could not be seen with an inch objective. Pittings, easily distinguishable after a little study, giving the quartz and the feldspar the appearance of having been etched very slightly with hydrofluoric acid, were quite visible. They were not uniform, but were sufficiently numerous in places to give the surface a greasy appearance. In some places the crystals of hornblende were completely gone, leaving the shape of their crystals in the rock. I did not notice any decay in any of the other minerals, and this one seemed to be most prominent in the vicinity of the vegetable growth, but I was unable to determine whether this appearance of a part of the quartz and feldspar, having gone into solution, was owing wholly or in part to it.

The quantity of the pieces removed from the Obelisk previous to putting on the paraffine is reported to have been as much as 800 pounds, and was probably considerably more than that. These pieces have been freely circulated, and can be frequently seen in the possession of curiosity hunters in different parts of the United States, some of them having a few pieces and others several pounds. What the future of the monument is to be is a matter of some anxiety. It is certain now that the opinion advanced at the time that the Obelisk was erected in Central Park, that the stone was quite sound when it was erected there, was a mistake. It was full of minute cracks when it came here. This is shown by the rapid disintegration of the stone, which has suffered more in the five years of exposure in the Central Park than in the 3 500 of its previous history, and also by the fact that the cracks are filled with vegetable life which received sufficient light through the stone to be colored yellow and green by the chlorophyl, the depth of the stone being shown by the variation of tinge of green. In

moist places this kind of vegetable growth does not take long to propagate itself, but in dry ones, like the interior of the cracks in granite, it must have taken many years. To have affected the stone even slightly, as it appears to have done, must have taken many more. It is quite possible that the germs of the vegetable matter were in the stone when it was in Egypt, and did not develop there because they did not have sufficient moisture to cause them to germinate, and that they only commenced to grow when brought here. It is also quite possible that this vegetable growth is not the cause of any of the apparent solution or pitting, though the appearances are exactly like those of other minerals whose surfaces have been slightly acted on; but it is certain that, while the chemical action on the stone is very slight, there has been a deep-seated and prolonged disintegration of the rock from purely physical causes, producing cracks and ruptures along the lines of greatest weakness, and that the stone has no longer the strength to resist the atmospheric influences of our very changeable climate, where the extremes of heat and cold are so very great, and where, as compared to Egypt, the amount of moisture in the air is always at a maximum.

The method of applying the present protecting (?) coating seems to have been a fatal mistake. Nothing of any account has been dissolved out of the stone, there is, therefore nothing to be replaced. If there had been, paraffine in solution would have been one of the best materials to fill them. Granite is not porous, there were, therefore no cavities to be filled. The stone being full of cracks from natural causes, the heat which was used to cause the paraffine to sink into the body of the stone, when applied to the outside would cause an expansion, which would not be responded to by the interior of the granite, and the cracks already there would increase in size, and pieces would chip off as they did, and new cracks would be formed in the stone, already weakened by long exposure. This is distinctly traceable on almost every specimen chipped off, the old part of the crack showing the stone somewhat less fresh on the surface, while the part recently cracked is quite fresh.

It is expected, now that the Obelisk is supposed to have been waterproofed, that the disintegration will cease, but this appears to me to be founded on an altogether mistaken theory, which is, that the cracking is alone due to the expansion of the ice formed in the cracks. The rapid and extreme changes of temperature in this climate in a stone, which,

from its mass alone, must have but a feeble conducting power, would be sufficient to cause the disintegration already begun, in a stone weakened by exposure to great heat in a dry climate, to continue with comparative rapidity without the intervention of ice, but simply from the continued expansion and contraction going on on its weakened surface. But in a moist climate like our own, where it was subjected to both extremes of heat and cold, it would take place rapidly, as it has done.

As it was a matter of interest to ascertain how far moisture had to do with the cracking under heat, I made the experiment of submitting pieces of granite which had become quite dry from having been kept housed many years, to as high a heat as could be obtained in the laboratory without melting, and to my surprise found that no spalling or even cracking occurred, although the pieces were subjected to the heat suddenly and for varying periods of time. It is well known that granite in buildings, when subjected to fire, spalls. This is owing to the moisture it contains; to the expansion of gas and liquids contained in microscopic bubbles in the quartz; and to the want of conductivity of the stone itself. Perfectly dry granite does not spall unless exposed very suddenly to a very high temperature. No granite, however, exposed to the weather in this climate is ever dry. Fresh granite contains about one per cent. of moisture. That weakened by age, like the surface of all the obelisks, may contain many times that amount, consequently all granites on the outside of structures do spall when exposed to fire. From the fact that the stone of the Central Park Obelisk is already weakened and probably full of fissures, which in this climate will tend to develop year by year, and from the very fact that the disintegrated stone will absorb more moisture than stone which is fresh, it seems probable that no protection or coating given to the stone will arrest the process of disintegration already commenced in it, if it is left exposed. Even if the surface was entirely waterproofed, the cold of winter and the heat of summer would act below the surface both of the coating and of the stone, causing the coating to break or fissures through it to occur, so as to let in the moisture, and then both causes would operate together as before. But in any case heat and cold will act altogether independently of moisture, whether the outside be coated or not, and further disintegration must take place under the same circumstances and conditions as that which has already so much weakened the stone. Placing the Obelisk in the Central Park, where it

is exposed to nearly every agency that could tend to destroy it; allowing the surface of a stone already so much weakened by disintegration to be heated, thus causing further cracks to be made in it; is a greater monument to public indifference and ignorance than the shaft ever was to the dignitary who first erected it, or the events chronicled in its hieroglyphics.

The same dangers, if the reports about it are true, threaten the Obelisk on the Thames Embankment, although not to the same extent. The climate of England is much less severe than ours, both in winter and in summer. The causes for disintegration being the same in both cases, will affect the London Obelisk less than ours, and there seems to be, so far as any examination of it has been made, no great present danger in leaving it exposed where it is. The Obelisk in the Place de la Concord, in Paris, is reported cracked all over its surface. Both the European Obelisks are therefore in danger of being seriously damaged within the next hundred years. Housing seems to be the only thing left for the Obelisk in Central Park.

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Vol. XV.—February, 1886.

FORMULAS FOR THE WEIGHTS OF IRON AND STEEL RAILWAY BRIDGES UNDER STANDARD SPECIFICATIONS.

By GEORGE H. PEGRAM, M. Am. Soc. C. E.

PRESENTED AT THE ANNUAL CONVENTION, JUNE 10TH, 1884.

WITH DISCUSSION.

It has been asserted that there is no law governing the variation of the weights of iron and steel bridges, and that, therefore, any formulas purporting to give them with exactness, or even approximate exactness, should be viewed with suspicion.

In the writer's opinion, this feeling at the present day is analogous to the opposition that is made to the use of steel for structural purposes, which does not always involve a proper consideration of the rapid progress which is being made in engineering.

The specifications for bridges and the details of construction are now comparatively uniform, and as the individual eccentricities of engineers become confined within narrower limits, the operation of a general law is proportionately extended.

The following specifications have been assumed. They might be called standard for this country, in the sense that they are the most usual.

Those affecting steel will doubtless be modified as our knowledge of that metal is increased.

Specifications.

The structure shall be proportioned to sustain its own weight, in addition to the moving load as hereinafter given for its particular class.

In order to provide for the effects of impact, the following percentage shall be added to the strains produced by the moving load considered as static:

For spans under 25 feet, 30 per cent.
" " of 80 " 8 "

The added percentage to vary between these limits, in proportion to the length of span:

End suspenders,	25 per cent.
Middle ties,	15 "
First counter-ties,	25 "
Second "	35 "
Last "	50 "

In plate girders the flange stress shall be considered as borne by the flange without the assistance of the web, and the shearing stress as borne by the web alone.

Under these conditions of loading, the stresses shall be limited as follows, viz.:

Wrought-Iron—

10 000 pounds per square inch in tension.
7 500 " " " " " shearing.
8 000 " " " " " compression,

reduced in columns according to the formula $\frac{8\,000}{1 + \frac{l^2}{a\,r^2}}$ in which

l = the length in inches.
r = the least radius of gyration in inches.
a = 18 000 for two pin ends.
a = 24 000 " one " "
a = 36 000 " two flat "

Lateral bracing shall be provided to resist a wind pressure of 30 pounds per square foot on twice the surface of one truss, together with a moving train surface averaging ten square feet per foot of span.

The stress in lateral rods shall be limited to 15 000 pounds per square inch, after adding 10 000 pounds per rod for adjustment.

Lateral struts shall be proportioned by the above column formula, with 10 000 in place of 8 000 as a basis.

STEEL—

Lateral rods,	20 000	pounds	per	square	inch.	
All other parts,	14 000	"	"	"	"	in tension.
	14 000	"	"	"	"	" compression,

except in members over sixteen diameters in length, where the column formula for wrought iron, with fifty per cent. added, shall be used.

With some engineers it is customary to vary the unit stresses instead of adding percentages. The result is practically the same, but the system of percentages seems to be more logical, and admits of varying stresses, closely approximating the effects of impact, which it would otherwise be difficult to express.

In the investigations of which this paper is the result, four classes of loading have been assumed, which will be denoted, for convenience, by the initial letter of the engine used.

They are:

Class M.—Two coupled Mogul engines, followed by a train weighing 1 820 pounds per foot of track, which was the common loading eight years ago, and is now used on some roads.

Class C.—Two consolidation engines, followed by a train weighing 2 240 pounds per foot of track, which has been the usual loading for the past five years.

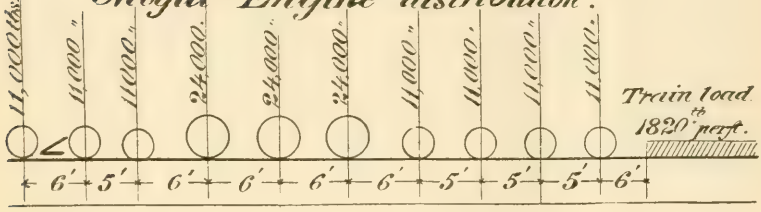
Class T.—Two "Typical" consolidation engines, followed by a train weighing 3 000 pounds per foot of track, which is now coming into use, and will probably be the loading for some time to come.

Class N.—Two narrow-gauge consolidation engines, followed by a train of 2 000 pounds per foot of track.

The distribution of weight in the several classes is shown in the following diagrams.

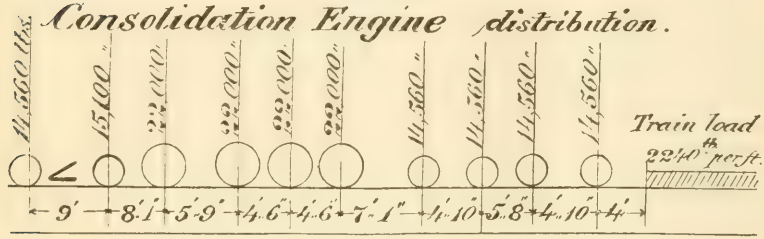
Class "M"

Mogul Engine distribution.



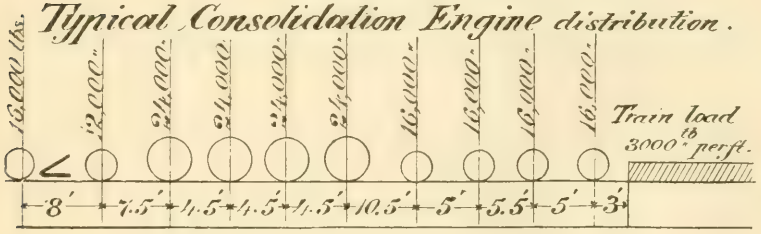
Class "C"

Consolidation Engine distribution.



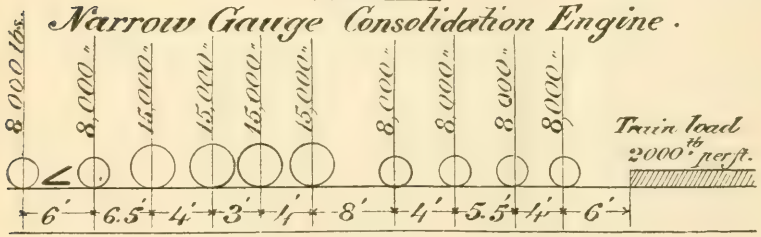
Class "T"

Typical Consolidation Engine distribution.



Class "N"

Narrow Gauge Consolidation Engine.



Under these specifications, the weights of bridges given in the following tables have been calculated with every attention to detail, and with a view of obtaining the most economical bridge in each case.

The aim has been to adopt a first-class construction, without any additions of doubtful utility, so that while a bridge might weigh more than that given through some special excellence, it should not weigh much less. This remark should be limited to spans under 300 feet, however, as above that the individuality of engineers becomes more marked, and so much difference of opinion exists as to the proper widths, etc., of bridges, that weights based on one kind of construction would be, in a measure, approximations for another kind.

The formulas herein proposed are intended to give the total or shipping weight of iron or steel in the span.

Some slight sacrifice of exactness has been necessary, in order to make the formulas as simple as possible.

All efforts to combine the total range of spans under one formula failed. It was necessary to have a formula for girders, and a different one for truss spans; or, what seemed more desirable, a formula which would include both girder and truss spans below 225 feet, which are of the most common occurrence, and a different formula for spans over 225 feet.

As far as possible the same style of formula has been maintained.

The type of bridge assumed is as follows: For spans below 75 feet, deck-plate girder bridges, 8 feet wide, connected with angle-iron bracing, and with cross-ties resting on the top chords; above 75 feet, through truss bridges, Pratt or single quadrangular trusses up to 150 feet, and Whipple or double quadrangular trusses over that length.

The widths assumed have been: For standard gauge spans under 255 feet, 14 feet in the clear; for 320 feet span, 18 feet center to center of trusses; for 420 feet span, 21 feet center to center; and for 520 feet span, 25 feet center to center. The floors of the spans consist of cross floor beams at the panel points, with a line of iron stringers under each rail, except for spans over 300 feet, which have three lines of stringers.

Differences in depth affect the weight less than would be supposed. Thus in a 60-foot girder span, for Class T, the difference in total weights between depths of 5 feet and 5½ feet was practically nothing, and for an 80-foot girder span, calculated for Class C, with depths of 6 feet, 6½ feet and 7 feet, the difference was less than 1 per cent. In a 180-

foot truss span, the difference in weights between depths of 26 and 28 feet was less than 2 per cent.

In a 520-foot steel span, for Class T, the difference in weight between a depth of 50 feet and one of 58 feet was about 3 per cent.; a depth of 56 feet was finally taken in this case.

The conditions on which the formulas are based have been stated. Modifications for other conditions may be made as follows:

If wooden stringers are used, deduct 195 pounds per foot for Classes M and C, 210 pounds for Class T, and 140 pounds for Class N.

For safety stringers add 100 pounds per foot for all classes.

For deck truss bridges add 10 per cent., and for double track bridges 90 per cent. to the formula weight.

Through plate girder bridges will not differ materially from deck bridges in weight, where the cross-ties are made to serve as floor beams; where, however, an iron stringer floor is used, it will be a close approximation to add 200 pounds per foot to the weight as given by the formula.

The following are the formulas proposed, in all of which

S = the span center to center of bed plates or end pins, as the case may be.

W = the total or "shipping" weight of iron or steel in pounds.

For iron bridges under 225 feet span:

$$W = \left(75 + \frac{S}{a}\right) S \sqrt{S} \qquad (1)$$

in which

$a = 7$ for Class T.
$a = 9$ " " C.
$a = 12$ " " M.

For Class N take three-fourths of the weight as given for Class T.

Table No. 1 is for this formula. In it are given the actual or estimated weights, and the corresponding weights by formula, with percentages of differences, taking the actual weight as unity.

TABLE No. 1.—For Iron Bridges under 225 feet span.

Span in feet.	Class M. Moguls and 1 820 lbs.		Per Cent. of Difference.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.	Class N. N. G. Consols. and 2 000 lbs.		Per Cent. of Difference.
	Actual.	$\left(75+\frac{S}{12}\right)S\sqrt{S}$		Actual.	$\left(75+\frac{S}{9}\right)S\sqrt{S}$		Actual.	$\left(75+\frac{S}{7}\right)S\sqrt{S}$		Actual.	$\frac{3}{4}\left(75+\frac{S}{7}\right)S\sqrt{S}$	
20	6 841	6 856	$-0\frac{2}{10}$	6 841	6 901	$+0\frac{9}{10}$	7 047	6 960	$-1\frac{2}{10}$	5 794	5 260	$-9\frac{2}{10}$
30	12 523	12 741	$-1\frac{7}{10}$	12 732	12 872	$+1$	12 979	13 036	$+0\frac{4}{10}$	9 576	9 828	$+2\frac{6}{10}$
40	18 635	19 794	$+6\frac{2}{10}$	19 283	20 072	$+4$	19 807	20 403	$+3$	14 832	15 303	$+3\frac{2}{10}$
50	27 639	27 997	$+1\frac{3}{10}$	28 561	28 456	$-0\frac{4}{10}$	29 115	29 022	0	22 303	21 765	$-2\frac{4}{10}$
60	37 360	37 200	$-0\frac{4}{10}$	38 590	37 944	$-1\frac{2}{10}$	39 502	38 874	$-1\frac{7}{10}$			
70	48 739	47 348	$-2\frac{8}{10}$	49 983	48 462	$-3\frac{1}{10}$	51 033	49 810	$-2\frac{3}{10}$			
*80	63 061	58 431	$-7\frac{3}{10}$	64 561	60 005	-7	66 061	61 793	$-6\frac{5}{10}$			
104	90 555	88 788	-2	93 050	91 812	$-1\frac{2}{10}$	96 957	95 312	$-1\frac{7}{10}$	70 464	71 484	$+1\frac{4}{10}$
130	124 871	127 200	$+2$	129 688	132 380	$+2$	137 285	138 850	$+1\frac{1}{10}$	99 638	104 145	$+4\frac{6}{10}$
150	154 427	160 781	$+4$	163 302	168 315	$+3$	173 849	177 190	$+2$	133 693	132 891	$-0\frac{6}{10}$
175	204 486	207 441	$+1\frac{1}{2}$	220 006	218 559	$-0\frac{7}{10}$	232 637	231 525	$-0\frac{1}{2}$	173 330	173 643	$+0\frac{1}{10}$
201.5	267 410	262 548	$-1\frac{8}{10}$	286 055	278 500	-3	304 459	296 300	$-2\frac{6}{10}$			
†255	441 065	391 755	$-11\frac{2}{10}$	468 884	420 668	$-10\frac{3}{10}$	508 415	453 776	-12			
†320	778 552	581 639	$-25\frac{3}{10}$	837 387	632 590	$-24\frac{5}{10}$	946 031	690 983	$-26\frac{9}{10}$			

* These 80 feet spans are plate girders. Pin connected trusses for this length would weigh less.

† These spans show how much the formula is in error for spans greatly exceeding the limit.

For Iron Bridges over 200 *feet span.*

$$W = \left(5 + \frac{S}{b}\right) S^2 \qquad (2)$$

in which $b = 100$ for Class C.

$b = 80$ " T.

Table No. 2 gives values for this formula.

TABLE No. 2.—For Iron Bridges over 200 feet span.

Span c. to c. in feet.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.
	Calculated	$\left(5 + \frac{S}{100}\right) S^2$		Calculated	$\left(5 + \frac{S}{80}\right) S^2$	
201.5	286 055	284 317	$-0\frac{6}{10}$	304 459	304 567	0
255	468 884	490 900	$+4\frac{7}{10}$	508 415	532 555	$+4\frac{7}{10}$
320	837 387	839 680	$+0\frac{3}{10}$	946 031	921 600	$-2\frac{6}{10}$
420	1 633 244	1 622 880	$-0\frac{6}{10}$	1 829 484	1 808 100	$-1\frac{2}{10}$
516	2 635 617	2 689 185	$+2$	2 977 191	3 048 631	$+2\frac{3}{10}$

For Steel Bridges over 300 *feet span.*

$$W = c\, S^2 \qquad (3)$$

in which $c = 6$ for Class C.

$c = 6.7$ " T.

Table No. 3 gives values for this formula. The formulas have been framed on independently calculated spans, in order to secure uniformity of conditions. Prototypes of the iron spans of 320 feet and under, with two exceptions, have been built within the past five years, and the formulas have otherwise been checked on over eighty spans built or designed within that time. The weights for steel spans are based on bridges which have been completely designed, but not built. They are of a very high type of construction. A frequent difference between the formula weights and those of existing bridges is caused by the custom which prevailed at one time, of assuming the webs of plate girders to assist the flanges and reducing the sections accordingly.

TABLE No. 3.—For Steel Bridges over 300 feet span.*

Span c. to c. in feet.	Class C. Consols. and 2 240 lbs.		Per Cent. of Difference.	Class T. Typ. Consols. and 3 000 lbs.		Per Cent. of Difference.
	Calculated	$6\ S^2$		Calculated	$6.7\ S^2$	
305.5	553 636	559 980	$-1\frac{3}{10}$	625 600	625 311	0
320	578 520	614 400	$-6\frac{2}{10}$	654 400	686 080	$+5$
374	884 700	839 256	$-5\frac{1}{16}$	944 000	937 169	$-0\frac{7}{10}$
400	974 243	960 000	$-1\frac{5}{10}$	1 073 545	1 072 000	$-0\frac{1}{10}$
420	1 052 720	1 058 400	$-0\frac{1}{2}$	1 182 835	1 181 880	0
450	1 237 540	1 215 000	$-1\frac{8}{10}$	1 382 500	1 356 750	$-1\frac{8}{10}$
475	1 333 830	1 353 750	$+1\frac{5}{10}$	1 481 510	1 511 687	$+2$
516	1 639 426	1 597 536	$-2\frac{7}{10}$	1 773 689	1 783 915	$0\frac{6}{10}$

* In these spans the floor-beams, stringers, lateral struts, counters and small lateral rods are made of iron, in accordance with the universal practice in this country.

DISCUSSION.

D. J. WHITTEMORE, Past Pres. Am. Soc. C. E.—In reference to this paper, it is pertinent to inquire whether there is such general uniformity of design at the present time among those members of our profession, expert in bridge construction, as to warrant the use of simple formulas in determining the approximate weight of metal that should be employed in their construction. Probably no other problem in mechanical construction has been subjected to such thorough analysis, and by so many able minds, as this of iron bridge design, and if, as a result of such rigid investigation, it is possible to estimate approximately the weight of properly designed structures by some such simple formulas as those given in this paper, I feel assured that to the engineer who has but little expert knowledge of the subject it will be often of great assistance and consequent value, though to the expert it may not afford that close approximation demanded by him.

Any general discussion of this paper should, and I trust will, cover the whole subject of bridge construction; in how far we agree as to standard train loads and wind strains, permitted strains in and forms of parts, and their mode of manufacture—all factors bearing upon the safe and economic weights of structures.

Applying the formulas to several structures, the weights of which are known to me, I find the same to give close approximate results, after deducting the weight of material used for purposes that are assumed by the writer to be of doubtful utility, but esteemed by me highly proper in resisting abnormal strains, to which every structure in constant use is liable.

It would be of interest, could we know whether the weights of structures given were derived from designs made mainly by one, or from designs made by several engineers of acknowledged experience.

I would ask our Members whether it is the generally accepted practice in designing plate girders to consider that the flange shall have sufficient area to bear its stress without assistance of the web, the latter to provide only for shearing stress as borne by the web alone, as stated by the writer.

Perhaps it may be proper, in the discussion of the paper, to call for an expression from our Members as to whether we are warranted in sub-

jecting steel to the strains specified by the writer, with our present limited knowledge of this metal, particularly when called to resist those of compression.

Considering the unusual facilities offered the writer in gathering data, and the labor and thought expended upon the subject treated, I trust this paper will receive that consideration it fully merits.

JOSEPH M. WILSON, M. Am. Soc. C. E.—The paper in question furnishes a valuable addition to what data we already possess on the subject of the determination of bridge weights. The ability to predetermine readily, and with reasonable accuracy, the weight which a bridge conforming to certain standard specifications should have, is a great desideratum. Not only is this so for the engineer, to enable him to assume his preliminary loading for calculation, but in this country, where the deciding of competitions and awarding of contracts is so often in the hands of those who have but little knowledge of the requirements for a good bridge, or what amount of material is necessary to build a bridge according to certain agreed specifications, the advantage of being able to apply readily some simple formula to the case is obvious. These formulas seem to meet the latter requirement particularly.

A paper was contributed some time since to the Institution of Civil Engineers, by Max Am Ende, on the Weight and Limiting Dimensions of Girder Bridges (Proc. Inst. C. E., Vol. LXIV, p. 243), which appears to consider the same subject, with rather a wider scope. Mr. Am Ende submits a general "theoretical formula for the weight, in which the weight of the bridge, the weight of the platform and the weight of the moving load; the principal dimensions and a number of constants, appear as undetermined quantities." The moving load and the weight of the platform being assumed, also the width of the bridge as a function of the span; and the constants being determined from the practice of the designer who constructs the parts of the bridge, the formula gives the desired result. Mr. Am Ende states that almost the only difficulty he encountered in the way of a satisfactory solution of the problem was in the case of secondary bracing.

The formulas of the present paper being derived from actual or calculated weights of known structures, or structures already designed, do not possess the scientific interest of the formula of Mr. Am Ende, and while no doubt giving reliable results within certain limits, yet they

might not be applicable for very long spans where little or no data exists to base them upon. Neither do they take any account of variations in the proportions of the structures. But they possess the great advantages of simplicity and readiness of application, both of which are very important in the uses to which they will probable be most frequently applied.

Should the standard loading for any structure in question not be the same as assumed in one of the cases in the paper, the requisite modification of the value of a can of course be made. In view of the continually increasing weights for engines and train service, the writer is in favor of high standards. The typical engine given in the paper is one which the writer had worked up several years ago, and adopted in specifications for bridges on the lines of the Pennsylvania Railroad, to anticipate what might come in the future. No engine has yet been built on this type, but one of another type, lately put in use, really surpasses it in its effects in certain special cases.

It appears to the writer that the results, as achieved by these formulas, might have been more readily accomplished graphically, by the use of a curve on cross-section paper, the ordinates laid out on the axis of the curve representing the spans of different structures, and the abscissæ the weights, either in pounds per lineal foot, or as desired. As many points for the curve as possible could be fixed from actual data, and the curve constructed thereon. Different classes of structures, such as solid plate girders, open-built girders, highway bridges, etc., could all be represented on the same sheet. The writer has used such a method with success, but while in his opinion better for the engineer, it might not, perhaps, be as applicable for the general public as formulas.

W. R. Hutton, M. Am. Soc. C. E.—Although no one series of formulas giving the relation of weight to span of railroad bridges, is in general use, yet many bridge-builders and experts have some rule by which to determine the weight of the structure for preliminary estimates with considerable accuracy.

The weight of the trusses will vary closely with the square of the spans, the floor system directly as the span and the width between trusses, and the horizontal bracing with the width and with the square of the span.

The specification quoted by Mr. Pegram contains one erroneous formula, although not uncommon in general practice. I refer to the application to a long steel column of the formula for an iron column "with 50 per cent. added." Experiments have shown that the resistance of long columns of steel varies with their length more rapidly than those of iron, approaching that of iron when the ratio of length to diameter becomes very great. This might have been anticipated with a metal whose resistance to crushing is much greater than iron, and its modulus of elasticity nearly the same. Those engineers act more prudently who limit the use of steel in compression to comparatively short parts, where its greater strength is available; using for the longer and generally lighter parts the better known material; for in these members the saving in weight is less, while the uncertainty as to strength is greater.

There is a specification that in plate girders the flange stress shall be borne by the flange alone, the shearing stress by the web alone. This is not unusual, but it would seem to be an unnecessary precaution in view of the low strains generally admitted in the metal of such forms.

The present dull season gives an excellent opportunity to many of our Members to work out a correct formula for strength of columns, in which the modulus of rupture (or the elastic limit) shall be combined with the modulus of elasticity—or at least a formula similar in form to Gordon's—which shall represent result of experiments in steel.

W. M. Hughes, M. Am. Soc. C. E.—Having the shipping weight of a large number of spans constructed from specifications substantially the same as those given in this paper, using two class "M" engines, one followed by the other, and this by a freight load of 2 000 pounds per lineal foot, I have applied the formula to a few of them with the result shown in the following table.

The iron stringers for 15-foot panels weighed about 160 pounds per lineal foot, a $\frac{5}{16}$ inch web plate being used. Where wooden stringers are used, I have deducted 195 pounds per lineal foot from result of formula as suggested by the author. As will be seen, the percentage of difference here in each case is minus. These are all "through" bridges, the last two being double track with two trusses. It will be seen that these weights are less than 60 per cent. added to the result of the formula, instead of 90 per cent. as suggested by the author of the paper.

It would seem that the difference here shown between the actual weights and those given by the formula are too great to make it of much practicable use.

TABLE No. 4.

Span.	Kind of Truss.	Stringer.	Depth.		Number and Length of Panel.			Actual Weight.	Weight by Formulas.	Per Cent. of Difference
Feet.			Ft.	In.	No.	Ft.	In.			
50	Pratt.....	Iron	6	3	4	12	6	31 360	27 940	— $10\frac{8}{16}$
61	"	"	8	0	4	15	3	39 294	38 150	— 3
75	"	"	21	0	5	15	0	51 157	52 812	+ $3\frac{2}{16}$
84	"	"	21	0	6	14	0	62 875	63 058	+ $0\frac{3}{16}$
105	"	"	20	0	7	15	0	78 476	90 115	+ $14\frac{9}{16}$
120	"	"	22	6	8	15	0	103 094	112 200	+ $8\frac{8}{16}$
135	"	"	22	6	9	15	0	124 015	135 110	+ $9\frac{5}{16}$
165	Whipple....	Wood	27	0	11	15	0	165 725	155 975	— $5\frac{9}{16}$
184	"	"	28	0	12	15	4	198 351	191 720	— $3\frac{3}{16}$
200	"	"	28	0	13	15	$4\frac{1}{2}$	240 241	225 240	— $6\frac{2}{16}$
150	Pratt.....	Iron	28	0	7	21	5	243 930	= formula.	+ $51\frac{7}{16}$
142	"	"	27	0	7	20	$3\frac{1}{2}$	235 700	= "	+ $59\frac{7}{16}$

Theodore Cooper, M. Am. Soc. C. E.—Every experienced bridge engineer has some method of approximating the weight of a proposed bridge. It may be by the use of formulas, by tables, by diagrams, or it may be by apparent guessing, based upon accumulated experience in similar work. Such methods used by any engineer for his own guidance, if given to the public, would be misleading; or, at the best, simple approximations, for they must be varied for each individual case, according to the specifications and requirements and to suit the particular style of bridge adopted.

The author speaks as if there was but one general standard of specification in use in this country. While in the better class of structures this is getting to be the case, in general terms there is still considerable variation in loadings, details and in the unit strains allowed on the various members. In general, a wider variation exists in the floor system than in the main girders. In some specifications the floor will be based upon a fixed load per lineal foot of track; in others upon some single type of engine; in others, upon the maximum due to several types of engines. In my own specifications I use a standard form of heavy freight engine, and a fixed load on two pair of axles to cover present or probable loads on heavy passenger engines. Some specifications call for four stringers under each track; specifications also vary in

he character of the wooden portion of the floor. In each special case there is an opportunity for variation, requiring the judgment of the expert to adapt the formulas.

The author of the paper speaks as positively of the allowed strains upon steel as in the case of iron bridges. Now, as before remarked, in the better classes of iron construction, experienced bridge engineers are approaching towards a general standard of unit strains for the various parts. But there is no justification for including steel in this statement. Our experience and knowledge of the adaptability of steel for the various parts of a bridge are still exceedingly limited. The assumption of 50 to 75 per cent. increase in unit strains upon steel over the allowed unit strains on iron members is entirely unwarranted as yet, and most certainly cannot be acknowledged as "standard for this country."

I must take decided exception to the inference conveyed in the first two paragraphs of the author's paper: That those engineers who are opposing the use of steel are in the same state of ignorance as those who assert there is no law (if there are any such) governing the variation of weights of bridges. Speaking for myself, I claim to have some "appreciation of the rapid progress being made in engineering;" nevertheless I do oppose the present loose manner of advocating the use of steel for bridges. This is called opposing the use of steel, but it is an entirely different thing. I believe steel to be the coming material of bridges and like heavy structures, but it will not become so until the manufacturers and users prove its capabilities for the particular purposes.

The steel men of the day are following blindly into the same paths that the iron men took when iron bridges were first built—making claims far beyond its capabilities, and using it in untried forms. The leading manufacturers of iron bridges soon found it necessary to determine by actual tests what different forms would best serve the purpose, and what were the capabilities of the material in its different forms. We hear no more of 60 000 pounds iron and flat plates for compression members.

Do the steel manufacturers appreciate that a false step can put back for years their possession of this great field of structural material? The danger is not in our great structures, built by intelligent and conscientious engineers, but from the thoughtless or unscrupulous imitators. Light structures are being built, in which the compression members, made of steel, are reduced to absolutely treacherous limits. Their ad-

vocates justify their proportions upon the loose statement that steel members are 50 per cent. stronger than iron. Now, if we may put this additional load upon tension members and compression members that must fail by crushing, there is no evidence that the same rule applies to members failing by deflection. In fact experiments show that at a point within the limits of actual practice, there is no increase of strength in steel over iron for long compression members.

As engineers, it is our duty to know the capacity of the materials we use, and we have the right to challenge these injudicious advocates for using steel in this manner, for their evidence of the increased strength of steel over iron in all its forms: actual tests, not assumptions.

The PRESIDENT—Gentlemen, you have heard the challenge. Bear it in mind.

CHARLES MACDONALD, M. Am. Soc. C. E —Have you any such tests in regard to iron ?

THEODORE COOPER, M. Am. Soc. C. E.—Yes, sir; we have full tests of all forms of iron—columns, eye-bars, etc., but in regard to steel we have nothing as to compression members, and it is simply presumed that steel, because it is steel—no other reason—is 50 per cent. stronger than iron. Now, if that is science, I fail to know what science is. I have always supposed that science was based upon actual facts, and experiments and deductions therefrom. Now, is it a scientific deduction that because steel is stronger in tension than iron, it is therefore stronger in compression ?

CHARLES MACDONALD, M. Am. Soc. C. E.—I should be sorry to see an impression prevail that American engineers were hesitating about the use of steel in bridge construction on account of a supposed lack of experimental information as to the strength or uniformity of that material. As a matter of fact, a very considerable number of tests have been made upon steel eye-bars of varying sizes, and the results prove beyond a question that they are being produced of a quality far superior to the best iron eye-bars. Even Bessemer steel, which has been thought inferior to open hearth in uniformity, is now giving quite as good results as the more favored material. In large sections to resist compression, as in top chords and end posts of bridges, we have no more experimental

information as applied to iron than steel, and we have a right to assume that where flexure does not result from compression, the strain per square inch on full sections in steel may be taken at the same ratio to the unit strain for that material, as would be considered good practice in the case of iron. In long columns, as in intermediate posts, it would be premature to advise the use of steel at present, by reason of the complicating influence of flexure. A comprehensive system of experiments in this direction is urgently needed, not only for steel but iron construction, and it is greatly to be deplored that the Government machine at Watertown, perhaps the only one in the country upon which such experiments could be made, is now fully occupied by what would seem to be very much less important investigation.

The floor system, composed as it is of plate girders, will not be made of steel until the price of steel plates and angles is reduced to an equality with iron, which must come at no very distant date.

THEODORE COOPER, M. Am. Soc. C. E.—My remarks have more of a bearing upon compression members than upon those for tensile strains. We have a very complete series of tests of iron columns of full-sized members. There have been made at Watertown a large number of tests of full-sized compression members, besides those made by Mr. Bouscaren, M. Am. Soc. C. E., and other individual tests.

These all show the reliability of the proportions adopted for our iron bridge columns.

They exceed the theoretical range allowed by Gordon's formula. But the only tests we have of similar forms of steel show less strength than iron columns. Some show only two-thirds the strength of similar ones in iron. I have carefully collected such information as possible as to tests of steel compression members, and I fail to find any justification for considering them as proportionately superior to those of iron. I would like to know from Mr. Macdonald if we are justified in using steel in the top chord of a bridge with 14 000 pounds compression per square inch, why iron of the same size and form would not be as good ? We have no evidence to the contrary.

Have you any evidence that iron in the same proportions would not be fully as strong. Is there any fairness in comparing iron bridges with 8 000 pounds on the top chords with steel bridges with 14 000 ? One or the other is wrong, in my opinion.

Mr. MACDONALD.—I think your point is very well taken.

GEORGE H. PEGRAM, M. Am. Soc. C. E.—Information has been asked on the process of development of the formulas. They are purely empirical; indeed I question the possibility of producing a formula based on theoretical considerations alone, which would be of any practical value. Such a formula would be very complicated and must contain constants, the values of which would have to be supplied from practice, and the formulas themselves checked on the actual weights of finished structures. One important element of uncertainty in a theoretical formula is the influence which the form of the truss has on the weight, and which would not be taken into account in the formula. The common forms of trusses—Warren, Pratt and Whipple—will all weigh about the same at 150 feet span for a single-track bridge. On both sides of this length they will differ. At 175 feet the Whipple truss will weigh 7 per cent. less than the Pratt, making a difference of about 4 per cent. in the total weights of the spans. Above this length the difference will increase.

As stated in the paper, all of the weights given were especially calculated, the weights of built structures and previously prepared estimates being used to check on. Having determined the minimum weights of first-class spans, the development of the formulas was a "cut and fit" process, subject to the conditions that no variable should appear but the span, and that this should be limited to the square and square-root, which may readily be obtained by the engineer from tables at hand, or in any case would be of easy determination.

Mr. Hughes has submitted a very interesting table of weights of structures which have been built.

Those of 50 feet to 84 feet truss spans are particularly interesting in comparison with the plate-girder spans of these lengths given in the paper. In the Whipple spans, if Mr. Hughes had used 160 pounds per foot for stringers, which he gives as the weight for 15 feet panels, the agreement with the formula would have been very close, the 165 feet span differing $2\frac{4}{10}$ per cent., the 184 feet being exact, and the 200 feet span differing but $3\frac{5}{10}$ per cent. The weights, however, are not sufficiently consistent to entitle them to use as a standard, and the conclusions drawn by Mr. Hughes would seem to have no basis in his table. For example, assuming the weight given for the 135 feet span as a basis,

a 142 feet single-track span would weigh 135 000 pounds, to which 75 per cent. must be added to give the weight of the 142 feet double-track span.

Assuming further that a 150 feet single-track span weighs 154 000 pounds, or 4 per cent. less than the formula, which would seem consistent with the other weights, only 60 per cent. must be added to give the weight of the double-track, 150 feet span. We have, therefore, in spans differing only 8 feet in length, 75 per cent. to be added in one case, and 60 per cent. in the other.

Without regard to formulas or tables, it would be a natural inference that a span of double the width to carry double the load should have double the weight, but, in fact, the larger sections, permitting higher stresses to be used, reduce the excess to about ninety per cent.

It is a source of regret that other tables have not been submitted, as in this way the variations that might be expected in the weights of different designs and under different conditions could be determined.

It is fair to ask what discrepancy should be allowed between the estimated and actual weights. In my opinion, the engineers' estimates, generally made before the structure is completely designed, should not be questioned when coming within five per cent. for ordinary spans. In checking on railroad bridges previously built, still greater variations may be expected. Railroads sometimes shorten a standard design without changing the sections, and sometimes lengthen it, increasing sizes, but without a proper reproportionment, in order to expedite the work.

It most frequently occurs, however, that a difference between the actual and formula weights, when checking on existing structures, is occasioned by the use of what has been termed "additions of doubtful utility," referring to stiff end bottom chords used in excess of the sections required, guard angles, devices for guiding a derailed train, etc. It was not meant to question the utility of these parts by what would now appear to be an unhappy expression, as I thoroughly agree with Mr. Whittemore in regard to them. But their use is not sufficiently general to be recognized in formulas such as are proposed, and it was thought best to let them be considered as extras.

Three elements of standard specifications which operate to limit the reduction of weight are as follows:

First.—No compression member shall have a length greater than 48 times its least diameter.

Second.—No plate in a compression member shall have a thickness less than one-thirtieth of the width between rivet supports.

Third.—No web plate shall have a less thickness than $\frac{3}{8}$ inch.

The latter cannot be said to be general, but is so frequently specified in first-class work, that it was observed in preparing weights for the tables.

Regarding steel, strictly speaking there are no standard specifications, but there is a considerable degree of uniformity in its use for the structures which have been built in late years, and which are in course of construction, and with the attention now directed to the subject, something like a standard practice will doubtless soon be developed. In the Plattsmouth and Bismarck Bridges, steel was taken at 15 000 pounds per square inch in tension, and the same in compression, for members under sixteen diameters in length. It was not allowed in long compression members. In other large bridges now in course of construction, steel is taken at 14 000 pounds in tension, the same in compression, for members under sixteen diameters in length, and the usual column formula for iron, with 37½ per cent. added for lengths exceeding sixteen diameters. This might properly be termed present practice. The only difference from the specifications given in the paper is in the case of members over sixteen diameters in length, where I have added 50 per cent. to the iron column formula.

What stresses it would be proper to allow in compression can hardly be determined from the meager data which we now have; but if 14 000 pounds can be used in lengths of sixteen diameters, fifty per cent. added to the iron column formula, which would correspond to a numerator of 12 000 in place of 8 000, would not seem excessive; in fact published tests would suggest that the numerator for the column formula be made the same as the unit stress for short members, as in iron, where we take 8 000 in both cases. And in support of this position I shall refer to the two tests published by Mr. George S. Morison, M. Am. Soc. C. E., in his book on the Plattsmouth Bridge, and the experiments of Mr. James Christie, M. Am. Soc. C. E., on Steel Struts, Paper 286, Vol. XIII of the Transactions of the Society.

What Mr. Christie has termed "hard steel" has 0.36 per cent. of carbon, corresponding almost exactly with the steel used in the bridges referred to.

In treating of bridges we should keep within the limits of the mem-

bers used in bridges, and it is of no importance how the material acts outside of those limits. A length of 16 diameters would correspond to about forty times the radius of gyration, and will not differ much from the usual top chords in steel spans. It is doubtful if members will ever be made less than 30 radii of gyration in length. The upper limit would be 48 diameters, corresponding to about 125 radii. This limit would never be exceeded and rarely reached. The question is: What will steel struts do between 40 and 125 radii in length? From the table given by Mr. Christie on page 265, Vol. XIII, as the results of his experiments, it appears that the excess of strength of hard steel struts over iron at 40 radii is 55 per cent.; at 80 radii, 65 per cent.; and at 120 radii, 52 per cent.

From the two tests made by Mr. Morison for Plattsmouth Bridge, which are especially valuable from the care with which they were made, and the minute description given of them in his published work, it appears that the same results cannot be expected in a riveted member that obtain in the single piece.

The steel was specified to contain 0.35 per cent. of carbon, and to have an ultimate strength and elastic limit of 80 000 pounds and 50 000 pounds per square inch respectively. One column, 34 radii in length, failed at 52 890 pounds per square inch, corresponding to the elastic limit of a tension member, and showed but 32 per cent. more strength than an iron column of the same size and construction, tested at the same time.

The other test was on an iron column of the same lateral dimensions, 9 radii in length, which failed at 46 474 pounds per square inch.

This latter column is shorter than would ever be used in practice, and the first is well within the limits of 16 diameters. It would therefore seem unwise to make a distinction between the stresses in long and short columns until it is developed by tests. It seems probable that new formulas will be required for steel, and that the stresses will be reduced.

Prof. A. J. Du Bois, Jun. Am. Soc. C. E.—While it would be incorrect to assert that there is no law governing the variation of the weights of iron and steel bridges, it would, I think, be safe to say that such law can only be imperfectly represented within very narrow limits by any formula in which the only variable quantity is the length of span. As the weight is affected not only by span, but also by depth, width, panel length, wind-bracing, flooring, and live load assumed, any

formula which assumes all these as constant, must be of very limited application. Mr. Pegram has but imperfectly met a portion of this objection by working out constants for three different cases of loading. Even then he finds it necessary to have a second formula for bridges over 200 feet span. Although his calculated results agree well with the actual, it is not apparent that they would agree equally well for different ratios of depth to span, and they certainly would not for varying floor systems and live load.

It has seemed to me that the preferable and more rational way would be to start with the floor system, and estimate its weight separately, as may be easily and accurately done, in any given case, according to the specification for the bridge in question. The wind-bracing can next be estimated. Then the weight of main girders. Lastly the total weight. A series of formulas, which will enable one to do this, would be free from the above objections. Such a method would enable a very close approximation to be made in any special case, according to the actual conditions of that case; and even for the same span, would be accurate enough to show the influence of any change in panel length, depth, live load, width, or floor system. Such a method would apply to all lengths of span, and to steel or iron, or part steel and iron or wood. I appreciate fully the labor and ability involved in Mr. Pegram's paper, and it is only fair to say that his formula seems to give better results than any of the kind heretofore proposed. I have thought, therefore, that the best criticism would be, to give in this connection a method such as that which I have alluded to.

This method involves the use of only two simple formulas. I have applied it to spans, actually executed, varying in length from 17-foot plate girders, to a 400-foot Pratt truss, with uniformly accurate results. Thus, for Bridge No. 44, Northern Pacific Railroad, by Kellogg & Maurice, 153 feet span, I find the weight 171 513 pounds, as against 158 493 actual, or 8 per cent. excess. For a span of same length in the Quincy Bridge, by Clarke, I find 150 612 pounds, as against 148 592 pounds actual, a difference of only $1\frac{2}{10}$ per cent. excess. Now, here are two bridges of the same span, but of very different construction, and our method takes account of such difference. Again, for span 5 of the Quincy Bridge, $197\frac{1}{3}$ feet span, I find 227 689 pounds, as against 224 400, a difference of $1\frac{4}{10}$ per cent. excess.

For span 1 of the Quincy Bridge, 247 feet span, I find 348 764 pounds,

as against 347 126, a difference of only $\frac{1}{10}$ of 1 per cent. For the Bismarck Bridge, by Morison, 400 feet span, I find by the same method 1 030 800 pounds, as against 975 356, a difference of 5 per cent. in excess. Yet this is part steel and part iron, and of very different construction from the Quincy Bridge. Again, for the Plattsmouth Bridge, by Morison, span 200 feet, I find 269 600 pounds, as against 270 387 pounds, a difference of only 787 pounds; while for one of the plate girders of the Plattsmouth Bridge Viaduct, span 30 feet, I find 3 719 pounds, as against 3 489, a difference of only 230 pounds.

If these examples can be taken as any indication, it would seem that the method leaves little to be desired. I am unable to apply it to any of Mr. Pegram's cases, as the necessary data as to depth, width, panel length, floor system, etc., are not known for the several cases. If he will apply it himself, I anticipate that he will report favorably. The method is neither tedious nor abstruse.

I will now give the formulas and apply them to the cases above cited, and then close by giving the development of the formulas themselves, showing that they are perfectly rational in form.

For plate girders—

Let W = the total external load in pounds.
W_1 = weight of a plate girder in pounds.
l = clear span in feet.
d = depth of plate girder in inches.
r = ratio of span to depth, both in same unit.
S = allowable unit stress, in pounds per square inch.

Also for trusses—

Let w_1 = total live load per foot of truss in pounds, for one truss.
w_2 = weight of floor, ties, planks, rails, spikes, etc.; also of floor beams, floor stringers, and wind-bracing per foot of truss, in pounds for one truss.
w_3 = weight of one truss in pounds per foot.
d = depth of truss in feet.
p = panel length in feet.

Then we have for the weight of a plate girder, consisting of riveted plate and angles, with connections,

$$W_1 = \frac{W\,l\,r + 2\,d\,S\,l}{1.2\,S - l\,r}$$

and for the weight of one truss in pounds per foot run

$$w_3 = \frac{(w_1 + w_2)\,(l - p)\,r + 20\,S\,d}{1.75\,S - l\,r}$$

These two simple formulas will enable us to estimate very closely, and with but little work, the weight of any iron, or iron and steel Pratt truss, double or single intersection, for single or double track, and for any panel length, depth or span.

We may estimate wind bracing by the formula, weight per foot run

$$\text{for wind bracing} = \frac{l}{2} + \frac{1600}{l}$$

EXAMPLE 1.—BRIDGE NO. 44, NORTHERN PACIFIC RAILROAD, BY KELLOGG & MAURICE.

Span = 153 feet; depth = 26 feet; width = 15 feet; panel length = 17 feet; $r = 6$.

In this case the weight of ties, planks, rails, spikes, etc. = 370 pounds per foot, or 6 290 pounds per panel.

The floor is supported by plate girder stringers and floor beams.

1. *Weight of iron stringers:*

$l = 17$ feet; $d = 28$ inches; $r = 7.3$; $S = 8\,000$ pounds.

The stringers were calculated for a live load = 51 000 pounds.

Weight of floor =	3 145	"
	54 145	" = W

Hence the weight of one stringer is

$$W^1 = \frac{54\,145 \times 17 \times 7.3 + 2 \times 8\,000 \times 28 \times 17}{1.2 \times 8\,000 - 17 \times 7.3} = 1\,512 \text{ pounds.}$$

1 620 = actual.

2. *Weight of iron floor beams:*

$l = 15$ feet; $d = 36$ inches; $r = 5$; $S = 8\,000$ pounds.

The floor beams were calculated for a live load = 81 600 pounds.

Floor	= 6 290	"
2 stringers	= 3 024	"
	90 914	"

Hence the weight of one floor beam is

$$W^1 = \frac{90\,914 \times 15 \times 5 + 2 \times 8\,000 \times 36 \times 15}{1.2 \times 8\,000 - 15 \times 5} = 1\,623$$

actual = 1 728

3. *Wind-bracing* $= \frac{153}{2} + \frac{1\,600}{153} = 86$ pounds per foot; actual = 77 pounds.

4. *Weight of main girder.*—The unit stress assumed is 10 000 pounds, and the live load 2 240 pounds per foot. Hence $S = 10\,000$, $w_1 = 1\,120$.

To find w_2, we have 8 floor beams at 1 623 = 12 984

18 stringers at 1 512 = 27 216

153)40 200

263 pounds per foot.

Wind-bracing 86

Floor 370

2)719

360 pounds = w_2

We have then $w_1 + w_2 = 1\,480$; and hence the weight of one main truss is

$$w_3 = \frac{1480 \times 136 \times 6 + 20 \times 10\,000 \times 26}{1.75 \times 10\,000 - 153 \times 6} = 386 \text{ pounds per foot.}$$

380 = actual.

5. *Total Weight of Iron.*—We have now for the two main trusses..... = 772

Wind-bracing..... = 86

Floor beams and stringers..... = 263

1 121

And hence total weight = 1 121 × 153 = 171 513

158 493 = actual.

Difference = 13 020 = 8 per cent. in excess.

EXAMPLE 2.—SPAN 7 OF QUINCY BRIDGE. BY CLARKE.

Span = 154 feet; depth = 22 feet; width = 14 feet; panel length = 11 feet; $r = 7$; weight of floor = 300 pounds per foot; weight of iron floor beams, 1 000 pounds; $S = 10\,000$; $w_1 = \frac{2\,500}{2} = 1\,250$.

Here, since the entire floor system is given, we have 26 floor beams at 1 000 = 26 000, or 169 pounds per foot.

Wind-bracing $= \frac{154}{2} + \frac{1\,600}{154} = 87$. Hence $\frac{169 + 87 + 300}{2} = 278 =$ w_2, and $w_1 + w_2 = 1\,528$. Hence

$$w_3 = \frac{1\,528 \times 143 \times 7 + 20 \times 10\,000 \times 22}{1.75 \times 10\,000 - 154 \times 7.} = 361$$

For the total weight we have then (722 + 87 + 169) 154..... = 150 612 pounds.

Actual..... = 148 592 "

Difference..... = 2 020 = $1\frac{3}{10}$ per cent.

EXAMPLE 3.—SPAN 5, QUINCY BRIDGE.

Span = 197⅓ feet; depth = 24 feet; width = 14 feet; panel length = 12⅓ feet; $r = 8.22$.

Same floor as preceding. $S = 10\,000$, $w_1 = 1\,250$, as before. 30 floor beams at 1 000 = 30 000, or 152 pounds per foot.

$$\text{Wind-bracing} = \frac{197.3}{2} + \frac{1\,600}{197.3} = 106 \text{ pounds per foot.}$$

$$\text{Hence } \frac{152 + 106 + 300}{2} = 279 = w_2, \text{ and } w_1 + w_2 = 1\,529.$$

$$w_3 = \frac{1\,529 \times 185 \times 8.22 + 20 \times 10\,000 \times 24}{1.75 \times 10\,000 - 197.3 \times 8.22.} = 448 \text{ pounds per foot per truss.}$$

For the total weight we have (896 + 106 + 152) 197⅓ ..... = 227 689 pounds.

224 400 = actual.

Difference ..... = 3 289 = 1$\frac{4}{10}$ per cent.

EXAMPLE 4.—SPAN 1, QUINCY BRIDGE.

Span = 247 feet; depth = 26 feet; width = 14 feet; panel length = 13 feet; $r = 9.5$.

Same floor as preceding. $S = 10\,000$, $w_1 = 1\,250$, as before. 36 floor beams at 1 000 = 36 000 pounds = 146 pounds per foot.

$$\text{Wind-bracing} = \frac{247}{2} + \frac{1\,600}{247} = 130 \text{ pounds per foot.}$$

$$w_2 = \frac{146 + 130 + 300}{2} = 288,\ w_1 + w_2 = 1\,538,\ w_3 = 568.$$

Total weight = (1 136 + 130 + 146) 247 = 348 764 pounds.

347 126 = actual.

Difference ..... = 1 638 = $\frac{4}{10}$ per cent.

EXAMPLE 5.—BISMARCK BRIDGE, BY MORISON.

Span = 400 feet; depth = 50 feet; width = 22 feet; panel length = 25 feet; $r = 8$.

Floor = 450 pounds per foot = 11 250 per panel.

1. *Weight of Iron Stringers.*—$l = 25$ feet; $d = 30$ inches; $r = 10$; $S = 8\,000$; live load = 50 000. Hence $W = 50\,000 + 5\,625 = 55\,625$, and $W^1 = 2\,770$ pounds. The weight, as computed from the dimensions, is 2 657 pounds.

2. *Weight of Iron Floor Beams.*—$l = 22$ feet; $d = 40$ inches; $r = 6.6$; $S = 8\,000$; live load $= 100\,000$. Hence $W = 100\,000 + 11\,250 + 5\,540 = 116\,790$, and $W^1 = 3\,281$ pounds. The weight, as computed, is 2 791 pounds.

3. *Wind-bracing* $= \frac{400}{2} + \frac{1\,600}{400} = 204$ pounds per foot.

4. *Weight of Main Truss.*—$S = 10\,000$, $w_1 = \frac{2\,000}{2} = 1\,000$.

15 floor beams at 3 281 = 49 215 pounds.
32 stringers at 2 770 = 88 640 "

137 855, or 345 pounds per foot.

Hence $w_2 = \frac{345 + 204 + 450}{2} = 500$ pounds, and $w_1 + w_2 = 1\,500$ pounds.

Hence $w_3 = 1\,014$ pounds per foot per truss.

5. *Weight of Iron.*—We have then (2 028 + 345 + 204) 400 ..... = 1 030 800 pounds.
Actual ..... = 975 356 "
Difference ..... = 55 444 = 5 per cent.

EXAMPLE 6.—PLATTSMOUTH BRIDGE, BY MORISON.

Span = 200 feet; depth = 30 feet; width = 16 feet; panel length = 25 feet; $r = 6\frac{2}{3}$. Floor = 460 pounds per foot = 11 500 pounds per panel.

1. *Weight of Iron Stringers.*—Same as before = 2 770.

2. *Weight of Iron Floor Beams.*—$l = 16$ feet; $d = 38$ inches; $r = 5$; $S = 8\,000$; live load $= 100\,000$. Hence $W = 100\,000 + 11\,500 + 5\,540 = 117\,040$; and $W^1 = 2\,005$ pounds.

3. *Wind-bracing* $= \frac{200}{2} + \frac{1\,600}{200} = 108$ pounds per foot.

4. *Weight of Main Truss.*—$S = 10\,000$, $w_1 = 1\,000$.

7 floor beams, at 2 005 = 14 035
16 stringers, at 2 770 = 44 320

58 355 or 292 pounds per foot.

Hence $w_2 = \frac{292 + 108 + 460}{2} = 430$ and $w_1 + w_2 = 1\,430$.

Hence $w_3 = 474$ pounds per foot per truss.

5. *Weight of Iron* —

We have then $(948 + 292 + 108)\ 200 = 269\ 600$ pounds.

$270\ 387$ actual.

787 = difference.

The above examples are sufficient to show the scope, ease and accuracy of the method. Let us now demonstrate the formulas used.

Demonstration of Formulas.

1. *For Plate Girders.*—The moment at center of span for a plate girder is $\frac{(W + W^1)l}{8}$. Dividing by the depth, we get the strain in a flange. Dividing again by S, we get the area of a flange; multiply by 2, and we have the area of both flanges.

Hence, since $\frac{12\,l}{d} = r$

$$\text{Area of flanges} = \frac{(W + W^1)\,r}{4\,S}$$

The weight per foot for iron is found by multiplying by $\frac{10}{3}$

Hence, the total weight of flanges $= \frac{(W + W^1)\,lr}{1.2\,S}$

The area of the web is $d\,t$, if t = thickness in inches. The weight per foot is then $\frac{10\,d\,t}{3}$, and

$$\text{Total weight of web} = \frac{10\,d\,t\,l}{3}$$

Add the weight of flanges and web and put the sum equal to W^1 and solve for W^1, and we have

$$W^1 = \frac{W\,l\,r + 4\,S\,d\,t\,l}{1.2\,S - l\,r}$$

Now, in practice, t is never less than $\frac{1}{8}$ inch, and seldom exceeds $\frac{3}{8}$ inch. To allow for connections, let us take $t = \frac{1}{2}$ inch, and we have

$$W^1 = \frac{W\,l\,r + 2\,S\,d\,l}{1.2\,S - l\,r} \quad \text{(I.)}$$

Which is the formula we have used for plate girders. It gives, as we have seen, in every case a very close estimate.

2. *For Main Trusses.*—The total weight on a truss, according to our notation, is $(w_1 + w_2)\,(l - p') + w_3\,l$, because one-half a panel at each end does not come on the truss. The moment at the center of span is then

$$\frac{[(w_1 + w_2)\ (l - p) + w_3\ l].\ l}{8}$$

Dividing as before by d and S, and multiplying by 2, we have for the weight of flanges per foot, $= \frac{(w_1 + w_2)\ (l-p)\ r}{A\ S} + \frac{w_3\ l\ r}{A\ S}$

where A is a constant. In the case of a plate girder A is 1.2, but for a truss with long panels A will be different, by reason of material required for stiffening long struts, and the fact that the area of flanges is not constant.

By comparison with executed examples, we find A to be for a Pratt truss about 1.75.

We have also for the web—weight of web per foot $= C\ d$, where C is a constant. Adding the weight of flanges and web, and equating to w_3 we have

$$w_3 = \frac{(w_1 + w_2)\ (l-p)\ r + A\ C\ S\ d}{A\ R - l\ r} \quad \ldots\ldots\ldots\ldots\ldots\ldots (II.)$$

From a number of calculated examples, we have determined the following values for A and $A\ C$.

	Pratt Truss.	Post Truss.	Triangular Truss.	Riveted Lattice.
$A =$	1.75	1.5	1.6	1.2
$A\ C =$	20	20	16	13

We have used the first values in all the preceding examples.

It is easy to deduce in similar manner for rectangular wood beams used as floor stringers,

$$W_1 = \frac{W\ l\ r}{3.55\ S - l\ r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (III.)$$

which may be used for pine stringers. Or generally

$$W_1 = \frac{W\ l\ r}{\frac{3}{4}\gamma\ S - l\ r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (IV.)$$

where γ is the weight of 12 cubic inches of the material.

It appears to me, from the accuracy of the preceding examples, that the method and formulas here given allow of a very accurate estimate of the weight of a bridge, whether railway or highway. The method is simple and rapid. It takes account of all the variables upon which the weight really depends, and accommodates itself to varying practice and specifications.

George H. Pegram, M. Am. Soc. C. E.—Prof. DuBois proposes to

determine the weight of a span by a series of formulas involving five distinct and rather laborious operations, which require the exercise of about as much skill as the ordinary detailed calculations. The formulas are admittedly approximations, and contain several constants derived from practice which it is not clear would be constant for all constructions, and the bases of which are not given. It is questionable whether it would not be better to spend more time and determine the exact weight in the usual manner. This will be governed somewhat by the degree of approximation obtained by the use of the formulas.

The examples given are not sufficient to show this. The first, a 153-foot span, for the Northern Pacific Railroad, differs eight per cent.; the next three are all spans from the Quincy Bridge, in which the weight of the floor is apparently taken from the bridge, the trusses and wind-bracing only being calculated by the formulas. These weights check well. The fourth and fifth examples are from the Bismarck and Plattsmouth Bridges, in which steel chords were used at a stress of 15 000 pounds per square inch, while Prof. DuBois seems to have taken the stress at 10 000 pounds. Their agreement with the actual weights would, therefore, rather disprove the formulas. The only examples which really support them are the three spans of the Quincy Bridge, which, as I have shown, are not completely worked out. The partial check on one bridge built so long ago is not conclusive.

The advantages which Prof. DuBois claims that these formulas have over those which I have proposed, are provision for the variations of the live load, the floor, the wind-bracing, the width, the panel length, and the depth. The present use of two engines followed by a train renders the live load a difficult element to provide for. Equivalent uniform loads for the same train may vary from 6 000 pounds to 3 000 pounds per lineal foot according to the length of span. To use a formula depending on a uniform load, it would be necessary to first determine what this load should be, which I cannot see that Prof. DuBois has attempted. In giving four classes of loading, I have endeavored to meet this requirement, for it is improbable that there will be any case of loading that cannot be brought under one of the classes, and in the case of a very different class of loading, a value for the constant in the formula may be readily determined. There is so much uniformity in the floor, wind-bracing and widths of bridges, that special calculations would seem an unnecessary refinement. The number of

panels within the ordinary limits will not materially affect the total weight. The depth is the only element of those given which would make much difference, and I believe that practice is sufficiently uniform to make it an unnecessary factor of the formula.

Prof. A. J. DuBois, Jun. Am. Soc. C. E. — I do not think that the five distinct operations involved in the method I propose will be found laborious compared to the ordinary detailed calculations. Three of these operations involve the determination of the stringers, floor beams, and wind-bracing. These can be easily and quite accurately determined in accord with any given specification, and as they make up more than half the dead load, it is advantageous to find them separately and not have them included in one formula whose only variable is the length. The weight of main girder is then easily found, and the fifth operation is simply summation of preceding results. This procedure is rational, and the formulas used are strictly rational in form. All of the varying given data have their due weight, and the result holds for the actual case in hand, not for one of four specifications. But two constants have to be determined, by using two given structures, and the method then holds good for single and double-track railroad bridges and for highway bridges as well, for any moving load which may be desired. It seems therefore, preferable to Mr. Pegram's in scope. Mr. Pegram's formulas are purely empirical, and limited by the specifications adopted and the class of structure, as well as by depth, width, panel length, etc.

In the sense that my two constants must be determined for the class of structure under consideration, my formula is approximate, but only in this sense. From two executed examples these constants are easily found. It is, I think, evident that these constants are the same for the same class of structure.

Mr. Pegram undervalues, I think, the examples given. The first two are bridges of the same span, but built under very different specifications. My method gives a close result for each, according to those specifications. Mr. Pegram's would give the same result for both, ignoring their difference. The weight of floor is taken as specified in each case, so also the live load in all the cases. The Bismarck and Plattsmouth Bridges show that it it not necessary to deduce new constants for part iron and steel. It would be out of place for me to determine equivalent uniform loads, because in each case I take an executed structure, assume the load

in accord with which the structure was designed, and compare the result with the actual weight. Mr. Pegram may assume any load he pleases, and my method will give him the corresponding weight in accordance with any practice as to floor system and wind-bracing. There is not, I think, such uniformity in depth as to make a formula which ignores it reliable in all cases, still less such uniformity in floor system and wind-bracing.

Mr. Pegram's method would require a series of formulas and constants to fit it to all the cases which mine easily covers. But I do not wish to be understood as undervaluing his formulas. Properly used within the limitations proper, they appear to be the best empirical formulas yet given. The objections I have stated are those common to all formulas of this character, and the advantages claimed for my own are those possessed by all rational formulas.

For a single-track railway Pratt truss, executed in accord with Mr. Pegram's specifications, his formulas will undoubtedly give readily a sufficiently accurate weight. Outside of those specifications, and especially for other classes of railway bridges, and for highway bridges, they have no application. I should be satisfied if, in this supplementary way, my method may be found serviceable, and if, as I have suggested, Mr. Pegram should think it worth while to test my method by the numerous cases he has already worked out, the necessary data for which are not available to me.

Geo. H. Pegram, M. Am. Soc. C. E.—I have checked Prof. Du Bois' formulas by the data on which my formulas are based with results as shown in the following tables:

TABLE No. 5.—For Plate Girders.

Span.	Depth.		Total Load, Exclusive of Girder.	Formula Weight of One Girder.	Actual Weight of One Girder.
Feet.	Ft.	In.			
*20	2	0	68 700	2 110	2 645
*30	3	0	96 000	4 594	4 871
40	3	6	110 000	7 772	7 818
40	4	0	110 000	7 554	7 824
50	4	6	130 000	11 786	11 835
50	5	0	130 000	11 550	11 822
60	5	0	144 000	16 714	16 432
60	5	6	144 000	16 307	16 614
70	6	0	156 870	21 962	21 117
80	6	0	172 000	29 446	28 535
80	7	0	172 000	28 097	28 114

* Without stiffeners, as these spans are sometimes built for stringers, the formula weight would be very close.

All weights and loads are in pounds. The stress is the average of the stresses for top and bottom flanges = 9 000 pounds. The live load is the equivalent uniform load for class T, with percentage for impact added as required by the specifications.

TABLE No. 6.—For Iron Trusses.

Span.	Depth.	Panel.	Stress.	Class of Live Load.	Load Per Foot, Exclusive of Truss.	Formula Weight of One Truss.	*Actual Weight of One Truss.	Per Cent. of Difference
Ft. In.	Ft. In.	Ft. In.						
104 0	24 0	17 4	8 800	*M*	1 820	34 112	23 658	$41\frac{6}{10}$
				C	2 000	34 632	24 458	
				T	2 090	34 798	25 074	
150 0	25 0	16 8	8 800	*M*	1 675	59 400	46 032	$22\frac{4}{10}$
				C	1 844	60 750	49 628	
				T	1 950	61 650	52 627	
201 6	28 0	16 $9\frac{1}{2}$	8 800	*M*	1 565	101 233	85 131	$12\frac{4}{10}$
				C	1 776	105 283	93 626	
				T	1 946	108 608	100 283	
320 0	34 0	20 0	9 000	*M*	1 605	267 680	273 671	$-6\frac{8}{10}$
				C	1 798	281 088	299 718	
				T	2 088	301 632	302 206	
255 6	29 0	18 3	8 900	*T*	1 943	168 364	185 558	$-9\frac{2}{10}$
	38 0	18 3	8 900	*T*	1 943	181 405	174 945	$3\frac{6}{10}$

* These weights do not include bed-plates, end-shoes and rollers, which for one truss will be: for 104 feet span, 1 800 pounds; 150 feet span, 2 000 pounds; 201½ feet span, 3 400 pounds; 255½ feet span, 3 900 pounds; and 320 feet span, 4 500 pounds. It is not clear whether Prof. Du Bois wishes these included, otherwise he makes no provision for them.

The formulas admit of much readier application than would be inferred from their complex appearance. The results for girders are all that could be asked. For trusses the formula gives very good approximate results for spans over 200 feet; but neither the girder nor truss formula gives proper variations for different depths and loads.

For example, an 80 foot plate-girder 7 feet deep, actually weighs 421 pounds less than one 6 feet deep, whereas the formula difference is 1 349 pounds; and a 255½-foot span, 38 feet deep, should weigh, by formula, 13 041 pounds more than one 29 feet deep, whereas it actually weighs 10 654 pounds less, a total difference of 23 659 pounds, or about 13 per cent.

As regards loads, the difference in weight between a 201½-foot truss for class T and one for class M, is by formula 7 500 pounds, while actually it is double that amount.

The 255½-foot span has been completely worked out for panels of 18.25 feet, 21.3 feet, and 25.5 feet respectively, and the greatest difference in total weights found to be about 1½ per cent.

The formula for bracing gives very good approximate results, except in girder spans, where any excess may be due in a measure to the angle-iron bracing. On the whole, I believe the formulas will be found very useful and sufficiently exact for the purposes for which they will generally be used, except in the trusses under 200 feet span, which, I should think, might be brought into line by some modification.

They do not, however, possess the advantages over mine which Professor DuBois claims in variations for depth, live load and panel length, and in conclusion, I would repeat what I have said on a former occasion, that the panel length will not materially affect the total weight, and there is not enough difference in practice in widths and depths to require that they should be factors in a formula for total weight.

J. S. Elliott, C. E.—Through the courtesy of Mr. A. M. Wellington, M. Am. Soc. C. E., who knew that I was collecting data on this subject, the advance proof of Mr. Pegram's article was kindly sent to me for purposes of discussion by Mr. John Bogart, Secretary of the Society. Mr. Pegram, by taking standard specifications, and by using for his three given span divisions precisely those types that both theory and practice unite in pronouncing most suitable, practically disarms criticism, and limits a discussion of his article rather to a checking of results and an

application to existing data. My object, therefore, is to see how applicable these formulas, derived from calculations of what weights should be according to the high standard just explained, may be to many kinds of railroad-bridge types built and in place.

Mr. Pegram's stipulations as to wooden stringers and deck trusses are taken into account in these comparisons.

The formula for class M, $W = \left(75 + \frac{s}{12}\right) s \sqrt{s}$ corresponds more closely than any other with the run of specifications of my different types. For the sake of condensation, only the variations in the form of a percentage are tabulated.

TABLE No. 7.—BRIDGES UNDER 200 FEET IN SPAN.

TYPE.	SPAN.	Variation from Actual Amount as a Percentage.	TYPE.	SPAN.	Variation from Actual Amount as a Percentage.
	Ft. In.			Ft. In.	
Plate Girder	23 0	+ 10	Pratt Ribbon	119 3	+ 2.7
" "	39 11	— 3	Through Pratt	126 0	— 7
Deck Pratt	44 4	— 8	Deck "	137 0	+ 1.3
Queen Post	50 0	+ 4.3	" Fink	150 0	+ 2.9
Deck Fink	50 0	— 7	" Warren	150 0	+ 2.6
" Pratt	61 0	+ 17.6	Through Pratt	153 0	+ 4.5
" "	62 6	+ 18.5	" Whipple	153 4	— 3
" Fink	90 2	+ 12.8	" "	153 4	+ 2
" Pratt	78 6	+ 16.4	" Pratt	154 0	+ 13
Through Pratt	80 0	— 7.6	" Whipple	162 3	+ 1.8
Deck "	98 8	+ 9.7	" "	170 0	+ 1.5
" "	99 9	+ 4.4	Deck Fink	180 0	— 4.3
" "	100 0	+ 9.2	Whipple Ribbon	181 6	— 15.5
" "	108 0	— 0.8	Through Whipple	187 6	— 2.4
" "	110 0	+ 5.3	Whipple Ribbon	189 0	— 11.8
Through "	111 0	— 1.5	Deck Whipple	195 0	— 2.4
Deck Bowstring	113 0	— 7	Through "	200 0	— 2.8
" Pratt	118 0	+ 9.5	Deck Fink	200 0	+ 1

Plus in the percentage column means that the formula gives in excess by the noted percentage; the reverse is the case when the minus sign is used.

This tabulated comparison gives 6½ per cent. as the average variation, but if we throw out, in conformity with Mr. Pegram's method, all truss types under 80 feet in span, and also the 154 foot through Pratt truss, as belonging to a railroad bridge built some twenty years ago, we reduce the variation to 5 per cent—a most gratifying result. Mr. Pegram's idea of adding 10 per cent. in case of deck types, is new to me. If anything, it would have been with me a case of subtraction. Small deck trusses

especially are narrower and shallower than through trusses of the same span. This means less material in floor beams, lateral bracing-posts and braces; but, of course, more in the chords. In the balancing, it would seem, then, that the result is 10 per cent. in favor of the deck variety. The few examples I have given in the tables seem to bear out Mr. Pegram in his opinion. The plus and minus variations for this type are pretty evenly divided. The rejection of the 10 per cent. addition would have made the percentage of error much larger and invariably a negative quantity.

Mr. Pegram has given a complete refutation to Mr. Stoney's dictum concerning the possibility of getting formulas applicable to bridge weights, and I regret not having at hand sufficient data to follow him in a suitable comparison with his results for double-track trusses and spans greater than 200 feet. Reliable data seem to be the hardest things to get quickly. It may not be entirely foreign to the subject to suggest that viaduct weights be given a like consideration. Working up some 34 viaduct quantities taken from the Cincinnati Southern Railroad Reports, I found that $W = (S + \Sigma)\ 250$, in which S = total length of viaduct in feet center to center, Σ = sum in feet of profile bent heights from top of masonry pedestals to top of columns, and W = weight of metal in pounds, gave variations of error, where 12 per cent. was a maximum, and 6 per cent. a mean. My formula gave 250 000 pounds less than the sum of actual amounts in the 34 structures (10 150 000 pounds) a variation less than 2½ per cent. These Cincinnati Southern Railroad Viaducts are of the ordinary American type, bents, mostly 30 feet apart, sometimes 60 and 45 feet. The most of them have about 80 feet as a maximum vertical height; the highest has 128 feet, and a significant variation error of —11 per cent., showing the necessity of a change in formula to suit great heights. The Kinzua Viaduct, for example, with its 302 feet maximum height, fell 33 per cent. short.

With more data, I think that an equation could be found to suit all sizes.

Geo. H. Pegram, M. Am. Soc. C. E.—I would ask Mr. Elliott whether he does not mean that Σ shall be the sum of the vertical heights of profile taken at some regular distance apart, regardless of what the spans of the trestle may be?

The formula assumes that the weight of bent per vertical foot is the

same as the weight of span per lineal foot. The span will actually weigh about 50 per cent. more per foot than the bent, but in trestles of the usual proportions it so happens that $\Sigma = 2\ S$ nearly, whence the shortage of one is made up by the overplus of the other, and the formula gives very good results.

In checking, I generally obtained very good results, except in low trestles or those with a short and deep profile. The formula is very simple, and in its present form would answer most purposes; but I would suggest that probably more latitude would be obtained with a modified form, viz.:

$$W = (3\ S + 2\ \Sigma)\ 110,$$

in which S, as before, is the total length of trestle, and Σ the sum of vertical heights at 30 feet distances, regardless of the length of span.

Mr. Elliott's extension of the scope of the paper to trestles suggests the idea of including draw-bridges as well. I think it will be sufficient to say that the total weight of a draw-bridge, including turntable, wheels and machinery to turn by hand, will be the same as that of a fixed span of the same total length, and proportioned for the same live load. I have tried this on a number of spans, from 150 feet to 400 feet, both single and double track, and find it remarkably exact.

W. H. Burr, M. Am. Soc. C. E.—Although the formula of Mr. Pegram is not at all a complicated one, it is probably possible to obtain another of a simple character, equally accurate, and possibly more rational. The floor system of stringers and floor beams is nearly a constant quantity for all ordinary spans, and any rational formula for the weight of such spans should contain a constant coefficient multiplied by the span. Below 250 feet of length, truss spans are also known to vary nearly as the square of the span. Hence, some such expression as Eq. (1) might be expected to give very closely the total weight of a bridge.

$$a\ S^2 + b\ S = W \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

a and b are constants, and S and W are the span in feet and total weight in pounds respectively.

A single illustration only of the application of this formula will be given.

The actual and computed weights of Mr. Pegram's "Class T" spans are given in the following table.

In this case $a = 6.25$ and $b = 260$.

TABLE No. 8.

Span.	Actual Weight in Pounds.	$6\frac{1}{4}\ S^2 + 260\ S.$	Per Cent.
Feet.			
20	7 047	7 700	+ 9.4
30	12 979	13 425	+ 3.6
40	19 847	20 400	+ 3.0
50	29 115	28 625	— 1.6
60	39 502	38 100	— 3.7
70	51 033	48 825	— 4.2
80	66 061	60 800	— 8.0
104	96 957	94 640	— 2.4
130	137 285	139 425	+ 1.6
150	173 849	179 625	+ 3.2
175	232 637	236 907	+ 1.8
201.5	304 459	306 159	+ 0.6
255	508 415	472 707	— 7.0

The different percentages for the plate girders are a little larger than those of Mr. Pegram, but for the remainder of the series they are somewhat less, with the advantage of a little more extended application. If the 60, 70 and 80 feet spans were lattice girders, as they should be for an economical construction, the variations would not exceed 3.2 per cent. for spans from 40 to 225 feet. Above the latter limit a break occurs in any simple formula that can be devised.

These few observations are made rather to draw attention to this simple formula than to cover the whole question.

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TRANSACTIONS.

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AN ACCOUNT OF SOME OBSERVATIONS OF STREET TRAFFIC.

By CAPTAIN FRANCIS V. GREENE, M. AM. SOC. C. E.

READ DECEMBER 16, 1885.

The object of this paper is to explain certain observations taken recently under the writer's direction, with a view to determining the actual and comparative traffic on the streets of certain cities; and also to call attention to the desirability of having the traffic systematically measured at frequent intervals, and on a uniform system, by the engineers having charge of the pavements in the various cities. In the twenty cities in this country which have a population of over 100 000 each (6 200 000 in the aggregate) the pavements represent an investment of about $250 000 000. With the single exception of railroads, it is doubtful if any other distinct class of engineering works represents so large an outlay. Whether or not these pavements properly fulfill the object for which they were designed, is a question of very great importance, affecting not only

the health and comfort of the residents of the cities, but also the whole subject of intra-urban transportation. This is a matter of dollars and cents, and a great many of them; for with bad pavements, where the resistance to traction and the wear and tear of vehicles and animals are great, the cost of transportation is often double what it is over good pavements, where the resistance to traction and the wear and tear are comparatively small.

Yet, although the pavement question affects directly such large property interests, I am not aware that any systematic observations have previously been taken in this country to determine the nature and volume of the traffic to meet which the pavements are constructed. In Europe such observations have been taken, though not on a uniform system. In the spring of 1873, Colonel Haywood made his well known investigation on "Accidents to Horses" in London, and for this purpose he was obliged to record the traffic. His observations were confined to five streets, Cheapside and The Poultry, Cannon, King William and Gracechurch streets, in the immediate vicinity of the Bank of England and London Bridge. The total length of streets observed was about one mile, and the number of stations fifteen. The observations were not simultaneous, but were scattered over a period of two months. Some of the stations were observed only one day, others two and three days, and some of them six days, which was the maximum. The observations were all made by the police, and were continuous from 8 A.M. to 8 P.M. There were two men at each station, and they observed for three hours consecutively, and were then relieved by others. Each man had two tours of three hours each, or six hours in all, each day. The exact manner in which the record was kept is not described in Colonel Haywood's report* further than the statement that it was kept in books specially prepared for the purpose.

Observations have also been taken on other streets in London and in Liverpool, but the method pursued in taking them I am unable to state. In Mr. G. F. Deacon's paper on "Street Carriageway Pavements," read before the Institution of Civil Engineers in 1879, he refers to certain streets of Liverpool having a traffic of so many tons per annum per yard of width, but he does not state how these data were obtained. And in

* Report to the Honorable the Commissioners of Sewers of the City of London on the Accidents to Horses on Carriageway Pavements. By William Haywood, Engineer and Surveyor to the Commission. London: Charles Skipper & East, Printers, St. Dunstan's Hill. 1873.

Mr. O. H. Howarth's paper on "Wood as a Paving Material under Heavy Traffic," read at the same time and place, he gives a tabular statement of the traffic on twelve principal streets in various parts of London. His unit for comparison is the tonnage per day per foot of width, which is evidently convertible into Mr. Deacon's unit by multiplying by 940. Mr. Howarth states that his observations were made in the month of November, 1878, and on every alternate half hour during the sixteen hours between 7 A.M. and 11 P.M. Further than this he gives no details.

In Mr. G. H. Stayton's paper, read in 1884, on "Wood Pavement in the Metropolis," is given a list of twenty-four streets (including those given by Mr. Howarth) with their traffic. He adopts as his unit the tonnage per yard of width in a day of sixteen hours, but does not state how the record was made.

In Paris, a series of careful observations were taken at 302 different localities between May 1, 1881, and April 30, 1882. The observations were continuous, night and day, for seven days at each place. Nearly all the places were intersections of streets, and the travel was counted both ways. The report does not state the manner in which the tally was kept. It is signed by MM. Allard and Barabant, the Chief Engineers of the two divisions of the city. The figures are very large, running as high as 33 000 vehicles and 42 000 horses at one place. No tonnage is given, the standard of comparison being the number of horses per meter of width.

The observations above noted—those of Haywood, Deacon, Howarth, Stayton and Barabant—comprise all that I have been able to find on this subject in print.

The observations which I shall now describe, were made under my direction during the months of October and November, 1885, by the employees of the Barber Asphalt Paving Company, which has an office and works in ten large cities; viz.: New York, Philadelphia, Chicago, Boston, St. Louis, New Orleans, Washington, Buffalo, Louisville and Omaha. In the arrangements for taking the observations, two objects were kept in view: First, to leave as little as possible to the judgment of the observer; and second, to make the record permanent, so that it could be preserved for examination in all its details at any time. For this purpose I drew up a blank form, a copy of which is here inserted.

TRAFFIC RECORD.

CITY.	STREET.	DATE.	OBSERVERS.	LOCATION OF OBSERVER.
.....	.....	188	.....	.....

HOURS.	VEHICLES.				ACCIDENTS.				REMARKS.
	Less than 1 ton.	Between 1 ton and 3 tons.	Over 3 tons.	TOTAL.	Falls on knees.	Falls on haunches.	Complete falls.	TOTAL.	
7 to 8.....									
8 to 9.....									
9 to 10.....									
10 to 11.....									
11 to 12.....									
12 to 1.....									
1 to 2.....									
2 to 3.....									
3 to 4.....									
4 to 5.....									
5 to 6.....									
6 to 7.....									
Total.....									

Width between curbs.....feet. Grade of street.....feet in a hundred $\frac{}{100}$

State of weather .....

Average temperature on pavement.....° Fahr. Length of street on which accidents are recorded.....feet.

Average speed of light vehicles.....miles per hour. Average speed of heavy vehicles.....miles per hour.

Personally appeared before me....., a Notary Public, in and for the.....of ....
in the State of....., the above named.....and....., who being by me duly sworn, do hereby certify, under oath, that the above record is correct to the best of their knowledge and belief.

.....188

....., Notary Public.

..... .

.....

TRAFFIC RECORD.

.....City.
.....Street.
.....188 .
.....Observer.

INSTRUCTIONS FOR KEEPING TRAFFIC RECORDS.

1st. Ordinarily two observers will be necessary to keep the record, in order that it may be uninterrupted during a period of twelve hours. They will divide the time between them according to their convenience, but should change with each other at the end of any given hour, and not during the hour. The observer should be stationed, if possible, in the second story window of a house from which he can have a clear view of the street for at least two hundred yards in each direction.

2d. The observer will be provided with an ordinary recording conductor's bell punch and a sufficient number of record slips in three colors, viz., white, red and blue, which will be used as follows:

WHITE, for vehicles less than 1 ton in weight.

RED, for vehicles between 1 and 3 tons in weight.

BLUE, for vehicles over 3 tons in weight.

The proper slip will be punched as each vehicle with its estimated weight passes the observer's station.

3d. In estimating weights, the following rules will be followed as a guide:

1-horse carriages, empty or loaded. 1-horse wagons, empty or light-loaded. 1-horse carts, empty.	LESS THAN 1 TON. WHITE SLIP.
1-horse wagons, heavy-loaded. 1-horse carts, loaded. 2-horse wagons, empty or light-loaded.	BETWEEN 1 AND 3 TONS. RED SLIP.
Wagons and trucks drawn by 2 or more horses, and heavy-loaded.	OVER 3 TONS. BLUE SLIP.

Special note will be made, in the column of remarks, of any unusually heavy loads, such as 6-horse trucks loaded with stone or iron, and an estimate given of their weight.

4th. Accidents will be recorded in the middle column of the proper slip, as follows:

WHITE, for partial falls on knees.

RED, for partial falls on haunches.

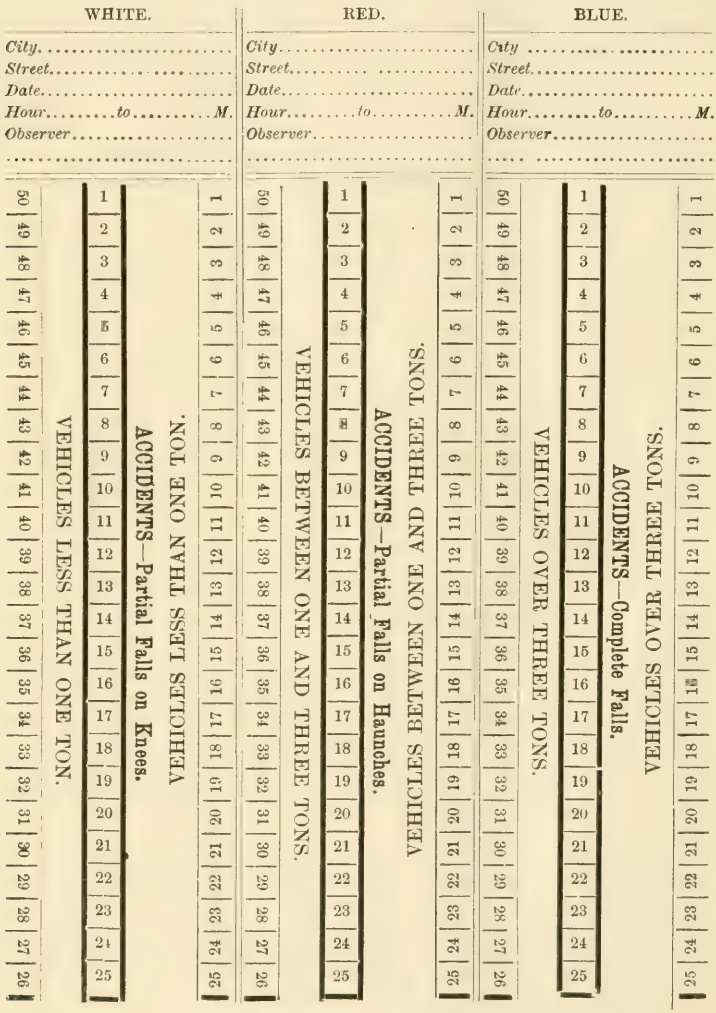
BLUE, for complete falls, in which the horse loses his legs entirely and falls on his side.

All accidents will be recorded which occur within a convenient distance of the observer's station—say to the end of the adjacent block on each side; and the total distance within which accidents are observed will be noted in the report.

5th. The state of the weather, condition of the pavement as to cleanliness, width of roadway between curbs, and location of the observer, should be noted on the report, together with a brief statement of any remarkable or unusual facts connected with the traffic.

6th. Each observer will make up his report on this form, giving the result of his observations, and mail it to Capt. F. V. Greene, West Point, New York, within twelve hours after the conclusion of each day's observations. The punched slips will be carefully preserved in the office of the agent until called for.

For making the actual count of vehicles, I thought the simplest and surest method would be to adopt the trip slip and punch system used by conductors of street cars for counting passengers. Three kinds of slips were printed, all of the same form, but each in a different color, and with different wording for the three classes of traffic and of accident. These slips are shown here.



The traffic was recorded in each city on these slips and blank forms, all of which have been preserved.

The printed instructions on the back of the record blank explain so fully the manner in which the record is kept, that it is hardly necessary to add anything to it.

The traffic is divided into three classes, light weight (less than one ton), medium weight (between one and three tons), and heavy weight (more than three tons); and in order to reduce the personal equation of the different observers to a minimum, the directions specify what classes of vehicles are to be counted in each class of weight. Nothing is then left to the observer's judgment and estimation except the question of "heavy" and "light" loads in one-horse and two-horse wagons. The result of different estimation in this respect between two observers would simply change a portion of the vehicles by one class, and the error in the final result could hardly exceed five per cent.

It will be noticed that the record includes the width and grade of the street, the length over which accidents are observed, the condition of the weather and the speed of vehicles; and that each day's record is signed and sworn to.

The agent in each city was instructed to select the three streets in that city paved with stone, asphalt and wood (if any existed), which, by common report, had the heaviest traffic in the class of pavement used on that street. For the purpose of making certain special comparisons, other streets were selected and their traffic recorded after the first reports were received. The record was in every case made on six consecutive days (Sundays omitted) at the same place, and it was continuous from seven A.M. to seven P.M., except when darkness prevented. No addition was made for this omission, no record was kept during the night, and no addition was made as an estimate of night traffic. In practice one observer was found sufficient on streets of very light traffic, and three (two of whom were always on duty) on the heaviest traffic, such as Broadway; on the majority of streets two observers were necessary, who relieved each other at the end of an hour or two hours.

These reports were all sent to me as soon as completed, and the results have been compiled by myself. In doing so I have discarded the weight of the horses altogether, not because they do not constitute a factor in the wear of the pavement, but because they were discarded in the English reports, and I desired as far as possible to make comparisons

with them. The addition that would have to be made if the horses were included would vary with the traffic. On streets where light vehicles predominate (as on Fifth avenue), it would be about eighty-five per cent.; on streets with heavy vehicles (such as Wabash avenue, in Chicago), it would only be about forty per cent.; and for other streets it would be between these two limits.

In digesting the reports the average daily traffic was obtained by dividing the total record for six days by six. To obtain the tonnage, I estimated the light-weight vehicles to average one-half ton each (including their loads), the medium weight two tons, and the heavy weight four tons. Multiplying the daily average of vehicles in each class by these figures and adding together the products, the total tonnage was obtained, and dividing this by the width between curbs, we get the daily average tonnage per foot of width. This is the standard or unit of comparison. It seems to me more convenient than Mr. Deacon's unit of the annual tonnage per yard of width, because the figures are much smaller and more easily compared and remembered, and it is also better than the French unit of the number of horses, because while the horses have their effect in the wear of the pavement, yet it is very much less than the wear due to the vehicles.

In this manner Table No. 1 has been obtained, which is the summary of the results at each locality.

In order to compare these results with those obtained in England, Table No. 2 has been compiled from the papers of Messrs. Haywood, Deacon, Howarth and Stayton. The Paris reports do not give any tonnage, and hence cannot be used for purposes of comparison.

TABLE No. 1.

GREENE ON STREET TRAFFIC.—TRAFFIC RECORD ON CERTAIN STREETS IN VARIOUS AMERICAN CITIES.

CITY.	VEHICLES. Less than 1 Ton.		VEHICLES. Between 1 Ton and 3 Tons.		VEHICLES. Over 3 Tons.		VEHICLES. Total.	TONNAGE. Total.	TONNAGE. Per Vehicle.	TONNAGE. Per foot of width	Pavement.	Width in Feet.	STREET.
New York	4 379	54 per cent.	2 502	= 34 per cent.	930	= 12 per cent.	7 811	10 905	1.39	273	Granite	40	Broadway, near Pine.
"	4 957	91 "	373	7 "	130	2 "	5 460	3 744	.68	94	Asphalt	40	Fifth avenue, opp. Worth Monument.
"	1 705	72 "	551	24 "	101	4 "	2 357	2 357	1.00	87	Granite	27	Wall, cor. Broad,
Philadelphia	3 454	58 "	1 499	24 "	1 128	18 "	6 081	9 237	1.52	142	Asphalt	65	Broad st., in front Pa. Railroad Depot.
"	3 160	63 "	1 403	29 "	446	8 "	5 075	6 302	1.24	97	"	65	Filbert st., in front City Hall.
"	1 280	70 "	424	24 "	110	6 "	1 814	1 928	1.06	74	Granite	26	Chestnut, corner 4th.
Chicago	907	25 "	1 966	52 "	824	23 "	3 697	7 561	2.08	151	"	50	Wabash, near Lake.
"	2 244	51 "	1 652	38 "	493	11 "	4 389	6 398	1.46	142	"	45	Clark, near Madison.
"	2 368	79 "	568	17 "	139	4 "	3 015	2 756	.90	77	Wood	36	La Salle, near Locust.
"	1 697	72 "	442	18 "	233	10 "	2 342	2 604	1.11	69	Asphalt	38	Dearborn, opp. Washington Park.
Boston	3 774	70 "	1 469	28 "	119	2 "	5 362	5 391	.99	196	Wood	27	Devonshire, opp, Post Office.
"	3 393	69 "	1 400	28 "	133	3 "	4 926	5 028	1.02	157	Granite	32	Devonshire, near Milk.
"	2 579	75 "	850	25 "	64	2 "	3 503	3 267	.93	126	Asphalt	26	Kilby, near State.
"	2 849	80 "	727	20 "	15	0 "	3 591	2 938	.80	75	Granite	39	Washington.
"	1 244	84 "	225	15 "	15	1 "	1 484	1 130	.79	43	Asphalt	26	Arch, near Summer.
"	993	89 "	118	11 "	3	0 "	1 114	744	.67	32	"	24	Court square.
St. Louis	2 278	69 "	697	22 "	292	9 "	3 247	3 691	1.13	103	"	36	Locust, near Beaumont.
"	1 900	65 "	753	25 "	288	10 "	2 941	3 618	1.23	72	Granite	50	Broadway, near Olive.
"	1 440	65 "	602	28 "	155	7 "	2 197	2 554	1.16	70	Asphalt	36	Pine, near Garrison.
"	765	74 "	164	15 "	117	11 "	1 044	942	.90	27	Wood	36	Chestnut, near Beaumont.
"	243	80 "	61	19 "	4	1 "	308	259	.84	7	Granite	36	Olive, near Beaumont.
New Orleans	1 468	42 "	1 284	37 "	726	21 "	3 477	6 204	1.81	155	"	40	Tchoupitoulas, near Poydras.
"	842	76 "	277	23 "	15	1 "	1 134	1 065	.94	35	Asphalt	30	St. Charles, near Washington.
Washington	3 133	70 "	1 196	26 "	191	4 "	4 520	4 622	1.02	66	"	70	15th, opp. Treasury.
"	1 517	79 "	369	19 "	48	2 "	1 934	1 688	.87	34	"	50	9th, bet. D and E.
"	1 287	79 "	305	18 "	48	3 "	1 640	1 445	.88	29	Granite	50	7th, bet. D and E.
"	902	71 "	297	25 "	56	4 "	1 265	1 289	1.01	21	Asphalt	60	6th, bet. Pa. ave. and B.
Buffalo	2 302	79 "	511	17 "	110	4 "	2 923	2 613	.83	47	Sandstone	56	Main, near Swan.
"	886	47 "	217	26 "	219	27 "	882	1 505	1.88	36	Asphalt	42	Main, near Bouck ave.
"	282	50 "	216	39 "	63	11 "	561	825	1.47	22	"	38	Linwood, near Ferry.
"	372	65 "	144	25 "	60	10 "	576	714	1.24	14	Sandstone	50	Main, near Glenwood
Louisville	1 929	58 "	1 226	36 "	190	6 "	3 345	4 176	1.25	69	Granite	61	Main near 3d
"	1 501	66 "	720	32 "	53	2 "	2 274	2 402	1.05	67	Asphalt	36	8th, near Walnut.
"	627	68 "	272	29 "	30	3 "	929	977	1.05	28	Macadam	35	7th, near Jefferson.
Omaha	4 406	66 "	310	6 "	36	1 "	4 752	2 967	.62	47	Asphalt	63	Douglass, near 15th.
"	2 339	95 "	134	5 "	3	0 "	2 476	1 449	.59	24	Granite	60	Farnham, near 14th
Average	1889	= 67 per cent.	734	26 per cent.	216	7 per cent.	2839	3267	1.15	77		44	

TABLE No. 2.

Traffic Record on Certain Streets in London and Liverpool.

City.	Vehicles.				Tonnage.					Street.
	Less than 1 Ton.	Between 1 Ton and 3 Tons.	Over 3 Tons.	Total.	Total.	Per Vehicle.	Per Foot Width.	Pavement.	Width in Feet.	
London	.....	.....	.....	12 148	13 507	1.11	**422**	Asphalt.	32	Gracechurch street.
"	.....	.....	.....	15 513	16 484	1.06	**412**	Wood.	40	King William street.
"	.....	.....	.....	8 167	8 330	1.02	**378**	Asphalt.	22	The Poultry.
"	.....	.....	.....	16 208	13 596	.84	**367**	Wood.	37	Strand and Fleet street.
"	.....	.....	.....	14 306	14 380	1.01	**322**	Macadam.	45	Parliament street.
"	.....	.....	.....	16 886	17 076	1.01	**300**	Wood.	57	Oxford street.
"	.....	.....	.....	9 419	9 260	.98	**290**	Asphalt.	32	Cheapside.
"	.....	.....	.....	6 128	7 588	1.08	**253**	"	30	Leadenhall street.
"	.....	.....	.....	10 776	9 358	.87	**253**	Macadam.	37	Piccadilly.
"	.....	.....	.....	12 132	10 658	.88	**242**	Granite.	44	Euston road.
"	.....	.....	.....	.....	.....	....	**216**	Wood.	..	Brompton road.
"	.....	.....	.....	6 371	6 506	1.02	**203**	Granite.	32	King William street.
"	.....	.....	.....	8 212	8 376	1.02	**195**	"	43	Edgware Road.
"	.....	.....	.....	10 796	9 668	.90	**186**	Macadam.	52	Regent street.
"	.....	.....	.....	.....	.....	....	**156**	Wood.	..	King's road.
"	.....	.....	.....	6 040	5 780	.96	**145**	Macadam.	40	Victoria street.
"	.....	.....	.....	.....	.....	....	**93**	Wood.	..	Sloane street.
Liverpool	.....	.....	.....	.....	.....	....	**382**	Granite.	..	(Street not named.)
"	.....	.....	.....	.....	.....	....	**232**	"	..	"
"	.....	.....	.....	.....	.....	....	**231**	Wood.	..	Great Howard street.
"	.....	.....	.....	.....	.....	....	**100**	"	..	Bold street.

Note.—The streets marked Macadam in the above list have all been repaved with wood since the date of Mr. Howarth's paper, owing to the expense of maintenance, which in some cases exceeded $1.00 per yard per annum. Edgware road and Euston road have also been paved with wood in place of the granite, on account of the noise arising from the latter.

The London records were not classified according to weight of vehicle, and hence I am unable to give the proportions of light, medium and heavy-weight vehicles. Colonel Haywood's report, however, classifies the vehicles according to the number of horses attached to them, and the proportion on King William street (wood) is 66 per cent. one horse, 31 per cent. two horses, and 2 per cent. three or more horses. The average of all the streets observed by him gave 71 per cent, 27 per cent. and 2 per cent. in the three classes. Mr. Howarth gives the number of cabs, omnibuses and loads above 3 tons on the streets named in his paper. On Piccadilly, for instance, with 10 776 vehicles:

6 084 (56 per cent.) are cabs averaging ½ ton each.

3 448 (32 per cent.) are omnibuses averaging 2 tons each.

728 (7 per cent.) are loads above 3 tons.

The great proportions of cabs and omnibuses explains at once the heavy traffic on London streets as compared with those of large American cities. In London no street railroads are allowed on any of the streets named in the above list, nor anywhere in the vicinity of them.

If there were street cars on these streets or in their vicinity, at least 80 per cent. of the omnibuses and 50 per cent. of the cabs would disappear.

This would take away 53 per cent. of the number of vehicles and about 70 per cent. of the tonnage. The tonnage on Piccadilly would be reduced to about 2 500 in place of 9 358, and 68 tons per foot of width in place of 253.

These figures show at once (what every one has noticed recently on Broadway) that street cars are a great relief to crowded streets, and that they effect a large saving in the wear of pavements. As laid in America, with T-rails and wooden sleepers, they are a nuisance to every other class of vehicle. As laid in Liverpool and London, with grooved rails and cast-iron sleepers bedded in concrete, they are almost unobjectionable. Vehicles pass over the rails without noticing them, and the only inconvenience arises from the fact that the cars cannot turn out in passing, and all other vehicles have to turn out for them.

In examining the figures of American traffic, we find that the average daily tonnage per foot of width is 77, and that it varies from 273 tons on Broadway to 7 tons on a granite street in St. Louis.

On some streets the traffic is remarkably uniform, and does not vary on any day more than 3 per cent. from the average; on other streets

the traffic is very irregular, being on some days as much as 50 per cent. greater or less than the average. In some cities bad weather makes a reduction in the volume of the traffic, and in other cities it does not. There seems to be no general rule for this. In regard to the hours of the day, the records everywhere show the same result—light traffic early in the morning and late in the afternoon; the maximum about 10 to 11 in the morning and 3 to 4 in the afternoon, and a decided falling off during the dinner hours, from 12 to 2.

As a general rule, the greatest traffic is on the business streets, but this is not always the case. In St. Louis, for example, Locust street is a purely residential street, leading from the business section towards the suburbs. Since it was paved with asphalt, its traffic has increased to 103 tons per foot of width, which is nearly 50 per cent. more than the traffic (72 tons) on the granite pavement of Broadway, the great business street of the town. Locust, Pine, Chestnut and Olive streets are all parallel and adjacent. There is nothing in their situation to indicate why the traffic should go to one of them more than to the others. They have been paved in the last few years—Locust and Pine, with asphalt; Olive, with granite; and Chestnut with wood on a concrete foundation. Their traffic is 103 tons on Locust, 70 on Pine, 27 on Chestnut, and 7 on Olive. In other words, the asphalt streets seem to have drained nearly all the traffic from the others. Similar results have taken place in Omaha. Farnham and Douglass streets were paved in 1882. The former being the main business street, it was thought necessary to pave it with granite, while Douglass street was paved with asphalt. The vehicles, however, choose the smooth pavement, and the traffic has been reversed, until now Douglass street has just twice as much as Farnham street. Similarly in Washington: Seventh street has been the principal business street for thirty years, and for that reason was paved with granite. Yet its traffic is now less than on the asphalt of Ninth street, which, when the two streets were first paved, was comparatively untraveled. Similar results have followed in Buffalo on the corresponding parts of Main street and Linwood avenue. It will be noticed that the wood pavement on Devonshire street, in Boston, supports a traffic of 196 tons—second only to Broadway.

The asphalt on Kilby street, in Boston, has a traffic of 126 tons, which is much larger than Washington street, in the same city (75 tons); Chestnut street, in Philadelphia (74 tons); Main street, in Buffalo (47

tons); Broadway, in St. Louis (72 tons); Main street in Louisville (69 tons); Seventh street, in Washington (29 tons); and Farnham street, in Omaha (24 tons), each of which is the main business street of its own city, and has been paved with granite under the supposition that its traffic was too heavy for asphalt.

The average tonnage per vehicle is an almost infallible indicator of the character of the street, *i. e.*, whether devoted to residential or business purposes. It ranges from .68 on Fifth avenue to 2.08 on a portion of Wabash avenue, in Chicago. The same character is indicated by the proportions of light and heavy vehicles on the street. On Fifth avenue, for instance, 91 per cent. of all the vehicles weigh less than 1 ton, while on Wabash avenue only 25 per cent. of them have so little weight. The general average is as follows: Less than 1 ton, 67 per cent.; between 1 and 3 tons, 26 per cent.; more than 3 tons, 7 per cent.; which agrees very closely with the proportions of one, two and three-horse vehicles observed in London by Colonel Haywood.

The table also shows that neither the maximum traffic nor the average traffic in each city, so far as here observed, is proportional to the population, although, as was to be expected, the largest cities have the largest traffic. In the five larger cities named in the above list, the average tonnage is 4 061, and 107 tons per foot of width; in the five smaller cities the average tonnage is 2 263, and 48 tons per foot of width. The streets in the larger cities have an average width of 37 feet, in the smaller cities 49 feet. The importance of laying out streets of proper width is clearly shown. Comparing, for instance, Wabash avenue with Devonshire street in Boston, we find the total tonnage on the Chicago street is 7 561 tons, while in Boston it is 5 362; yet the Boston street has to support 196 tons per foot of width, against only 151 tons in Chicago, because the width of the former is only 27 feet, while the latter has 50 feet.

The average tonnage per foot in each city, so far as here observed, varies from 151 in New York to 30 in Buffalo, and the general average is 77. A great many more observations would have to be taken, however, before attaching much importance to these averages. Mr. Deacon, in discussing the cost of maintaining various pavements, reduces the cost in each case to a standard traffic of 100 000 tons per yard per annum, which is very nearly the same as 100 tons per foot per diem. Mr. Stayton uses 150 tons per diem as a standard of average traffic. Both of these

writers assume that the wear of the pavement is a direct function of its wheel-tonnage per unit of width. It is doubtful if the problem is so simple as this; there are other factors, such as horse-tonnage, grade, climate, proportion of light and heavy vehicles, etc. But it is certainly true that the wheel-tonnage is the most important factor, and is almost the only one which can be readily stated in figures. Hence it is approximately accurate to assume that a pavement of the same materials, laid in the same manner, should wear twice as rapidly under twice the tonnage per unit of width. With this assumption, both Mr. Deacon and Mr. Stayton compare the maintenance cost of various pavements reduced to the same traffic standard, *i. e.*, the standard being 100 tons, if the pavement has 200 tons traffic per foot, its maintenance cost is divided by ½, and if it has only 50 tons, it is multiplied by 2 before comparing them, in order to see which is essentially the cheapest and most durable pavement.

This is, of course, the practical object of recording the traffic on the street. The first cost of a pavement is only one of the factors which determine its cost in a number of years. To determine this it is necessary to take account of the cost of maintenance and the interest on first cost. And in order to determine the inherent merits of the pavement, the volume of traffic which causes the expense of maintenance should also be taken into account, as has been done by the English engineers above mentioned. It is needless to say that there are no statistics of the cost of maintenance in this country that would enable any such comparisons to be made. Any one who doubts the utility of such records and comparisons has only to contrast the miserable condition of the vast majority of American streets as compared with those of England, where the paving question has been scientifically studied by means of such records, in order to have his doubts removed.

In compiling the reports of accidents, I have followed the method adopted by Colonel Haywood, *i. e.*, the number of horses is multiplied by the distance over which they are observed, in order to obtain the total distance traveled. This is divided by the total number of accidents in each class in order to find the distance traveled before an accident occurs. This is the standard of comparison. The accidents are divided into three classes, viz.: Falls on knees, falls on haunches and complete falls. The number of horses was not actually counted, but was obtained by estimating the light-weight vehicles to have one

horse, and the medium and heavy weight two horses each. This gives a result which is probably below the actual number of horses.

The result of the observations compiled in the manner indicated, is given in Table No. 3.

The general result of these observations—which it should be noticed were made on 192 days, in thirty-two different streets of ten different cities, and include 807 552 horses and 81 051 miles traveled—is as follows, viz., a horse will travel the following distance before meeting with an accident of any kind:

On asphalt	583	miles.
On granite	413	"
On wood	272	"

Colonel Haywood's results were as follows:

On asphalt	191	miles.
On granite	132	"
On wood	330	"

The result of both sets of observations thus accords in showing that there are fewer accidents on asphalt than on granite. They do not accord in regard to wood, the London experiments showing it to be the least slippery and ours the most slippery of all the three classes. On account of the small number of our observations on wood, it is hardly fair to make the comparison, but there are very few cities in which wood is now used in America, and hence a greater number of observations could not be obtained. In the above statement it will be seen that there are about three times as many accidents on the London pavements as on the American. This is due partly to the dampness and fogs of London and partly to the greater traffic, and consequently greater amount of manure. The manure and fog combined make up the "grease" from which the London streets are seldom free, although their system of street cleaning is, I think, the best in the world. The fine grained Neuchatel asphalt, which is used in London, is also much more slippery than the gritty Trinidad asphalt used in this country.

In further classifying the accidents, we obtain the Table No. 4:

TABLE No. 3.

GREENE ON STREET TRAFFIC.—ACCIDENTS TO HORSES. 1.—ASPHALT PAVEMENTS.

	Horses.				Accidents.				
	Number Observed.	Average Load.	Average Distance in Miles.	Total Distance Traveled in Miles.	Falls on Knees.	Falls on H'nches.	Complete Falls.	Total.	Street.
New York	35 778	.62	.028	1 002	2	1	2	5	Fifth avenue.
Philadelphia	55 248	1.03	.133	7 348	0	9	10	19	Broad street
"	41 940	.91	.176	7 380	0	0	0	0	Filbert street.
Chicago	17 922	.80	.113	2 025	0	0	0	0	Dearborn street.
Boston	26 562	.76	.077	2 045	10	5	4	19	Kilby street.
"	10 344	.66	.078	807	1	1	2	4	Arch street
"	4 940	.60	.047	235	0	1	1	2	Court square.
St. Louis	25 416	.89	.113	2 872	6	0	0	6	Locust street.
"	17 724	.84	.113	2 003	0	0	1	1	Pine street.
New Orleans	8 646	.73	.113	977	1	1	1	3	St. Charles street.
Washington	35 442	.78	.076	2 701	6	0	3	9	Fifteenth street.
"	14 106	.71	.076	1 072	1	1	0	2	Ninth street.
"	9 768	.80	.038	371	0	0	0	0	Sixth street.
Buffalo	7 548	1 20	.680	5 131	0	0	0	0	Main street.
Louisville	18 282	.80	.113	2 066	0	0	1	1	Eighth street.
Omaha	30 588	.58	.119	3 392	0	0	0	0	Douglass street.
Total	360 254	.....	.....	41 427	27	19	25	71	
Distance traveled before an accident occurs. Miles	.....	.....	.....	.....	1 534	2 180	1 647	583	

2.—GRANITE PAVEMENTS.

New York	67 458	.97	.057	3 846	14	2	0	16	Broadway.
"	18 055	.78	.057	1 029	7	1	1	9	Wall street.
Philadelphia	14 488	.80	.113	1 590	0	0	1	1	Chestnut street.
Chicago	38 205	1.19	.113	4 317	0	0	0	0	Wabash avenue.
"	39 084	.95	.113	4 416	0	0	1	1	Clark street.
Boston	38 754	.78	.071	2 748	8	0	0	8	Devonshire street.
"	26 006	.66	.064	1 664	10	1	0	11	Washington street.
St Louis	23 952	.90	.113	2 716	8	0	2	10	Broadway.
New Orleans	32 916	1.13	.057	3 990	17	1	4	22	Tchoupitoulas street.
Washington	11 958	.72	.076	909	4	1	0	5	Seventh street.
Buffalo	21 264	.73	.113	2 403	0	0	0	0	Main street.
Louisville	28 566	.88	.113	3 230	0	0	1	1	"
Omaha	15 678	.55	.119	1 865	0	0	0	0	Farnham street.
Total	376 384	.....	.....	34 723	68	6	10	84	
Distance traveled before an accident occurs. Miles	.....	.....	.....	.....	510	5 954	3 472	413	

3.—WOOD PAVEMENTS.

Chicago	21 972	.75	.113	2 483	0	0	0	0	La Salle street.
Boston	41 700	.76	.045	1 876	10	4	0	14	Devonshire street.
St. Louis	7 242	.78	.075	542	2	1	1	4	Chestnut street.
Total	70 914	.....	.....	4 901	12	5	1	18	
Distance traveled before an accident occurs. Miles	.....	.....	.....	.....	408	980	4901	272	

TABLE No. 4.—Distance Traveled (in miles) before an Accident Occurs:

	Falls on Knees.	Falls on Haunches.	Complete Falls.	Accidents of any kind.
Asphalt	1 534	2 180	1 647	583
Granite	510	5 954	3 472	413
Wood	408	983	4 901	272

From which it appears that in falls on knees the order of excellence is asphalt, granite, wood; in falls on haunches, granite, asphalt, wood; and complete falls wood, granite, asphalt. This shows that while a horse will bark his knees more frequently on granite than on asphalt, yet he will be thrown, by pulling him up suddenly and throwing his hind legs from under him, more frequently on asphalt than on granite. Nearly every "complete fall" on asphalt is reported as due to this carelessness in driving. On the other hand the accidents on granite are largely due to bad blocks. Even in the newer granite block pavements the mistake is made of using blocks nearly five inches in width. If the horse misses his hold on the first joint his foot is pretty sure to go out from under him. In Manchester and Liverpool, whose granite pavements are the admiration of every one who has seen them, the blocks average only three inches in width; and the horse has a chance to catch on the second joint if he slips from the first.

For the purpose of comparison, the Table No. 5 is given, showing the percentage of each class of falls, as taken from Colonel Haywood's report, and from my own observations:

TABLE No. 5.

	London.			American Cities.		
	Falls on Knees.	Falls on Haunches.	Complete Falls.	Falls on Knees.	Falls on Haunches.	Complete Falls.
Asphalt	32 per cent.	24 per cent.	44 per cent.	38 per cent.	27 per cent.	35 per cent.
Granite	46 "	8 "	46 "	81 "	7 "	12 "
Wood	85 "	3 "	12 "	66 "	28 "	6 "

The principal difference in these two tables is the much greater proportion of complete falls on the granite pavements of London in comparison with those of America. The reason of this is not clear. It may be due to the fact that a horse who loses his fore-feet on the London streets is apt to go down altogether on account of the "grease," whereas he may recover himself on the American streets.

Colonel Haywood's report classifies his accidents according to grade of street, cleanliness of street, state of the weather, rate of travel and method of harnessing. Into these refinements I have not attempted to enter. The only classification I have made is according to the nature of the pavement. With one exception the grades here observed were all flat, *i. e.*, less than $2\frac{1}{2}$ in 100, and no distinction was made for weather or cleanliness.

The general result of the observations, both in London and in this country, shows that there is much less difference as regards slipperiness between asphalt and granite pavements than is generally supposed; that for falls on the knees granite is more slippery than asphalt, while for falls on the haunches asphalt is more slippery than granite; and that, without classifying the nature of the fall, about 40 per cent. more horses fall on granite than on asphalt. It is needless to remark that these facts are quite opposed to the popular opinion on the subject.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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REMARKS ON THE AERATION OF WATER.

BY CHARLES B. BRUSH, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE 26TH, 1885.

The following statement relates to the process for the aeration of water, as introduced during the past year on the works of the Hackensack Water Company, and as now being introduced in the City of Philadelphia. The Hackensack Water Company, reorganized, supplies that portion of New Jersey on the west bank of the Hudson River, lying opposite that portion of the City of New York above Grand street, and including Hoboken, Union Hill, Weehawken, North Bergen, Ridgefield, Hackensack and other adjacent places. The water is taken from the Hackensack River at New Milford, about five miles above Hackensack; from thence it is pumped 14 miles through a cast-iron main to the principal reservoir on the heights north of Hoboken. The Hackensack Reservoir is connected with this main about three miles from the pumping station.

During June, 1884, an unpleasant taste and smell was first noticed in the water furnished to Hoboken; in July these peculiarities became

very pronounced, and then a green scum began to collect on the water in the Hoboken Reservoir. After a while this took the appearance of green paint. The reservoirs were full in anticipation of the summer drought. There was no unpleasant taste or smell from the water drawn along the force main; none from the water in the river; none in the water supplied to Hackensack; and none in the Hackensack Reservoir; but after the water was delivered into the Hoboken Reservoir the taste and smell became offensive. As soon as this green scum appeared on the surface of the water in the Hoboken Reservoir, the water in that reservoir was cut off from the city, and Hoboken was supplied directly from the pumps. I found that by keeping the water in motion from the time it left the river until it was delivered to consumers in Hoboken, the unpleasant taste and smell largely disappeared. The assistance of Dr. Leeds, of Stevens Institute, was then obtained. We found that this green scum continued to increase on the surface of the water in the reservoir until a white scum appeared in the middle of it, with a further development of bluish streaks and a foam-like appearance, culminating finally in white patches over much of the bluish-green surface. This remarkable appearance was confined to the surface, the water 2 feet below not being visibly affected. When the wind sprang up this discoloration disappeared. Analyses, frequently made, showed that there was a deficient supply of oxygen in the water, and that this development of green *algæ* increased as the supply of oxygen in solution in the water decreased. In its normal condition the amount of oxygen in solution in good water is about 6½ cubic centimeters per liter, $\frac{6.5}{100}$ of 1 per cent. by volume, but in this case it had run down to about 3½ cubic centimeters. The drainage area of the works is about 100 square miles. There is no sewage pollution in this area. The difficulty was entirely of vegetable origin. Since the deficiency in oxygen, together with a somewhat large percentage of dissolved extractive matters of vegetable origin, were the only abnormal features revealed by chemical analysis, Dr. Leeds suggested that we could improve the water by supplying the oxygen requisite to bring the water to its normal condition, and probably succeed in oxidizing the dissolved extractive matters at the same time. He had found by laboratory experiment that he could cause the offensive taste and smell which affected the Philadelphia water supply as taken from the Schuylkill River in January, 1883, to entirely disappear, and had been led to devise a process by

which aeration could be easily applied. The benefit of aeration of potable water has indeed been recognized from time immemorial, and had already been made the subject of certain patents in this country by Mr. R. d'Heureuse, but these involved the use of air at merely ordinary atmospheric pressure. Dr. Leeds controls these patents, but has improved upon them by introducing the air under greater pressure, which not only causes the work of oxidation to be very rapidly and effectually performed, but makes the process of such a character as to be easily applied in practice. Under his advice we set up air compressors at New Milford, and forced air into the mains under a pressure of about 125 pounds to the square inch. By so doing the oxygen in the water is increased, and, ordinarily, when the water is not turbid from suspended earthy matter (a difficulty encountered after heavy storms, and which, of course, can only be completely removed by filtration), it manifests a sparkling appearance and has only a pleasant taste and smell. The water as drawn from the main is often perfectly white, but in a moment it clears up from the bottom like soda water, and those who take the water directly from the main drink it with delight while still effervescent. We commenced in September, 1884, with this aeration process. We had no difficulty during the fall and expected none; but during the winter there was a taste and smell in the water which was supplied to Hackensack. There was some 40 or 50 days' supply in the reservoir of that town, which was covered with ice 2 or 3 feet thick, and all the streams that supplied our pumping station were covered with ice. In order to test the aeration theory, we pumped air directly into the distribution main at Hackensack; within six hours the smell and taste had disappeared from the water in these mains. Then we cut the ice and pumped air into the water in the reservoir, after which the difficulty there also disappeared. On May 30th, 1885, the smell and taste again appeared in the water supplied to Hoboken. We were partially aerating the water at the time; we increased the amount of air supplied and the smell and taste entirely disappeared. At present the condition of the water seems to be excellent. We are, it is true, just entering our dangerous season. Perhaps the aeration process will not carry us through, but we believe it will. We are careful to keep the water in our reservoirs so moving that every part shall be supplied with aerated water. We also keep the reservoirs properly stocked with fish, principally carp and black bass, removing them as they become too numerous. Analyses of the water from different points

are made once a month, and sometimes oftener. Microscopic examinations are made also, which show that the animal life is changed in different conditions of the water.

DISCUSSION.

Col. WILLIAM E. MERRILL, M. Am. Soc. C. E.—Drinking water is presumably affected by sunlight. I observed some years ago that all the reservoirs in Paris were entirely covered, so that the water was kept in the same condition as it would be in any subterranean reservoir, and it occurred to me that many of these vegetable organisms that appear on water and cause this taste and smell are due to the vivifying action of the sun. I would like to ask whether the supply to Hackensack and Hoboken is exposed to the sunlight.

Mr. BRUSH.—Most of it is exposed to the sunlight. The flow of the river is from 20 000 000 to 600 000 000 gallons per day, and the worst stage is at its time of least flow. I believe that the offensive condition of the water is due to the slow flow and the decomposition of vegetation in the river. There is not vitality enough in the water to resist, burn up, or throw off the impregnating effect of decomposing vegetation. As long as the water is actually in motion it does not stagnate and ferment; but will do so soon after coming to rest. If I am right in this view, I do not see that any practical gain will be obtained by covering the reservoir; this operation will always be expensive, and often impracticable. If the water contains sufficient oxygen in solution, I believe that the more sun, air and motion we give to water the better it will be.

Col. MERRILL.—That is so contrary to European practice, that I think it ought to be discussed.

J. F. FLAGG, M. Am. Soc. C. E.—This scum is vegetable matter?

Mr. BRUSH.—It is the result of vegetation.

Mr. FLAGG.—Is it growing vegetable matter?

Mr. BRUSH.—No, it is not growing. It looks like green paint.

Mr. FLAGG.—Do you suppose it is decomposed?

Mr. BRUSH.—Yes, in some process of decomposition. I take it that the wild plants along the river would naturally improve the water, yet when the water gets to that slow stage of flow, vegetable life appears which is injurious. This vegetation does not appear while the water is kept in active motion.

JAMES B. FRANCIS, M. Am. Soc. C. E.—What is the depth of the reservoir ?

Mr. BRUSH.—The reservoir is not large. The main reservoir is 16 feet deep; the small one 13 feet.

Mr. FRANCIS.—Is there any marked difference in the water as you go down ?

Mr. BRUSH.—I do not allow much difference to appear, because I keep it in constant motion. After that trouble at the Hackensack River I found that the reservoir at Hackensack, which was built long ago, before I had anything to do with it, was built in such a way that the lower eight feet had no circulation and the water was taken directly above that, and my impression was that the taste and smell came from that lower portion. I drew that all off and there has been no complaint received since.

Mr. FRANCIS.—I would like to say to Col. Merrill that the covering of the reservoirs in Europe has a different object in view from the one he mentioned, namely, to protect the water from dust and soot. Many of them are near cities where dust and soot accumulate very rapidly.

Col. MERRILL.—That no doubt is a very important object. But is it possible for any organism to grow in water unless you have some ozone ? In an artesian well you have pure water—nothing can happen to it; no vegetable organism can appear. Now, if you put water where no sun can get at it, is it not likely to be pure ? Can anything live without sunlight ? It seems to me that if Mr. Brush would combine his method of aeration with this method, keeping the water away from the influences that cause life, then the combination would be perfect.

J. N. TUBBS, M. Am. Soc. C. E.—I agree to a certain extent with Colonel Merrill. I am connected with water-works in Rochester, N. Y. Of course these odors in water are produced by a variety of causes. Our experience was something like this. Our water supply is taken from a mountain lake about seven miles long and one-half to three-quarters of a mile wide. It is carried in a close conduit of iron and discharged into a reservoir, and then carried nine miles and discharged into another reservoir. In 1876, the first year the works were opened, a very disagreeable smell, denominated "fishy," was discovered in the water, and it continued during the months of September and October, and even later. The conduit then extended out into the lake itself about 600 feet by a flexible-joint pipe three feet in diameter and opening three or

four feet from the surface. The trouble continued for two or three months. In the succeeding year this pipe was put into thirty feet of water; in the year after that some difficulty occurred at the Lake. I happened to be there, and it was evidently from vegetable growth that had reached maturity at the bottom and came to the surface of the Lake, perhaps by the propeller of a steamboat stirring it up. It had an appearance like fine brick-dust. There was no particular odor or taste to the water until the July sun came upon it, when the vegetable organisms seemed to decay, and it was the decay of these which produced the bad smell. This did not go down any great depth—two and a half feet, perhaps. All around the shore of the lake was that smell like decaying fish, but none of this reached the depth of thirty feet where the pipes were. In the construction of the works we had arranged to throw the whole supply of water into the air for the purpose of aeration. A twenty-four inch main is carried up through masonry and is discharged in twenty-one jets into the air under the head of the storage reservoir eight or nine miles away. It goes up into the air and is thoroughly aerated. This was done while the fishy odor was present, but while it did some good it did not seem to do much. I am of opinion that these vegetable organisms grow in patches in localities beneath the water, and when they reach the surface the sun causes them to die and then there is a disagreeable smell. I think that when this occurs no aeration will do much good, and the remedies must be applied to the source of the growth of these organisms; and I believe, further, that the suggestion of Col. Merrill, that the covering over of the water, without preventing the free access of the air, will, to a large extent prevent the decay and the disagreeable results following.

J. J. R. Croes, M. Am. Soc. C. E.—In some places where ground water is depended upon for the supply of water to towns—and is drawn from covered galleries, there has not, so far as I know, been trouble from the growth of *algæ;* but in a number of cases where pure spring water is received in open galleries, on exposure to the sunlight in the basins where it is received, nearly every summer the water becomes intensely disagreeable, getting a cucumber or a fishy taste. That has been the case in Taunton, Mass., where an open gallery was used. At certain periods of the year there has been trouble from the fishy taste and smell. During last winter that gallery was extended and a connection made with the river; not a pipe directly, but on the

river-side an embankment of coarse gravel and broken stone was built, through which the water percolated, filtering it to a certain extent. Just before the river water was introduced into the gallery, this smell and taste occurred in the old basin, but in a letter from the Superintendent, Mr. Billings, he said that since the introduction of the river water into the basins, part of the supply being taken from the ground water and part from the river, the taste and smell had entirely disappeared; that the mixture of the two waters had caused the vegetable organisms to disappear. I think that in no case where these underground basins are entirely covered, and the water protected from the air until it has reached the consumers, has there been any unpleasant odor or taste; but in every case where this underground water is exposed to the sunlight before it is pumped into the mains, something of the kind has occurred during the summer.

Mr. J. B. Francis.—The experience in Boston in the aqueduct might favor Col. Merrill's idea. Vegetable matter upon the aqueduct there, when examined, was found generally to be very dense at the commencement of the aqueduct, gradually declining until it disappeared at some distance. Whether that was caused by getting into the dark is a question.

Mr. Brush.—The water we supply to our high-service consumers on Union Hill was not exposed to the sun. It was taken directly from the main and stored in a covered tank open to the air. During our trouble the complaint from our high-service consumers was as emphatic as from those who were supplied directly from the Hoboken Reservoir.

Emil Kuichling, M. Am. Soc. C. E.—I have seen quite a number of these covered reservoirs and I have not been able to find that even when covered completely they serve to prevent the growth of vegetable matter. A case in point is one at Hanover, a new reservoir. There was vegetable matter occurring there which I saw myself, a fine species of *algæ* without much color. Biological researches have shown that the color of these vegetable organisms is dependent upon the amount of light they receive. Some species of *algæ* were cultivated in the laboratory at Leipsic in a glass tube, and as they exposed this glass to the light, it developed according to the intensity of the light a red color or a green color upon the *algæ*. In some cases they were simply colorless and yet they could be detected. In Berlin they have a series of wells along the shore of Tegeler Lake, and in these

wells they discovered a few years ago a growth of vegetable matter which has been the subject of investigation by some of the most eminent scientists of Berlin. It grows in the filtering gallery and also in their force mains to such an extent as to almost close up the mains. The exclusion of light is therefore not sufficient to produce immunity from this vegetable growth. A number of other wells in Europe as well as in this country show growths of *algæ*. There is not so much smell or taste in waters thus stored, which is due perhaps to the fact that they are protected somewhat from the sun and from the effect of sunlight itself. The cases observed indicated a slight taste of the water, also a slight smell, but not sufficient to become offensive. Heat and light together will cause a different state of growth and development of these plants than if they were placed in abnormal conditions of darkness. The spores will get into water. No amount of filtration has sufficed to keep these germs out. The experiments of Professor Pumpelly for the National Board of Health two or three years ago, showed that these germs could still be cultivated and were in water after filtration through one hundred feet of sand. Now if we find these spores of vegetable matter in a well sunk into the subsoil of broad valleys, and also find them after filtration, in storage reservoirs, it seems to me that the question is still a debatable and an unsolved one how to treat them. The aeration of the water at Rochester, as passed up though the fountain of the distributing reservoir, had the effect of modifying the taste to a large extent, but standing to the leeward side of this fountain one could easily detect, three hundred feet from it, the peculiar fishy smell, although there was a considerable improvement in the quality of the water.

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THE ABT SYSTEM OF RAILWAY FOR STEEP INCLINES.

By WALTON W. EVANS, M. Am. Soc. C. E.

READ OCTOBER 21ST, 1885.

The problem of how to climb steep inclines on railways in the best and most economical way, has taxed the inventive powers and exercised the brains of many and clever engineers. A third of a century or more ago, almost all of the important valleys of the civilized world had been occupied by railways depending on weight and the adhesion of smooth surfaces as the chief elements of traction in moving railway trains. As the engine began to mount inclines, and as there was an ever-increasing desire to haul heavier trains, the locomotive was enlarged in all its capacities and made more ponderous. The rail had to grow in weight and size to be able to carry the locomotive and perform its part of the duty, and excellent as have been the results on easier gradients in the reduction of cost of transportation, when we come to steeper inclines the service fails in an alarmingly rapid degree. There the locomotive with its five pairs of wheels coupled and a total weight of 144 000 pounds, having 128 000 pounds on the driving-wheels, has become a perfect giant. Yet a giant who is chiefly seen wrestling with his own stupendous mass is a sore exhibit of impotence, and this brings us to a point where we may with propriety say, stop, we can go no further, we have come to

conditions where these Saturn-like monsters are devouring everything but the empty pockets of the shareholder. We know that there are rich valleys up in mountain districts; we know that there are rich veins of minerals up in mountain regions that the greed of man and his ever-restless march to the front is anxious to develop. There is a trite old saying that "Where there's a will, there's a way." It would be a sad thing if we had to cry "halt" in railway progress, and had arrived at a point beyond which we could not go. We should hail with joy any invention that bids fair to let us take another step in advance in solving this great problem of making the steel-shod horse ascend the inclines of a mountain side.

The writer has studied the inventions and improvements of Mr. Roman Abt, of Switzerland, and believes that he has done a great deal towards solving the problem we are interested in, and by so doing he has opened new fields for railway enterprise and for the athletic exercises of the iron horse. But before entering on a description of the Abt system of railway, and the machinery he uses to produce economical results, it is proper to review the past history of the designs and inventions connected with overcoming steep railway gradients. It is always fascinating to note how ideas spring up, prevail for a time, are finally rejected, to be taken up anew and stamped in the molds of advanced and progressive arts. Everything has its proper time. Slow is the course of true progress, but, "Chi va piano, va sano."

Historical Review.

When Trevithick, in 1804, had succeeded in building the first locomotive engine that ever pulled a train of loaded cars, an event that took place on the Merthyr Tydvil Railway in South Wales, he relied merely on the adhesion between his smooth driving-wheels and smooth rails for obtaining the requisite traction. His locomotive being light, the tractive power appeared to be very small, and yet the engine weight was more than the cast-iron tram-plates of those days could stand. Instead of grasping the proper remedies of greater engine weight combined with stronger rails—those remedies which led to the wonderful developments of later years—Trevithick, partly also because called away by other interests, gave up all further attempts to improve upon his locomotive, and expressed the conviction that nothing but some additional means of propulsion could yield satisfactory results.

A suggestion like this, coming from so prominent an engineer as Trevithick, did not fail to shape the course of inventive genius for the following years. But nothing of a practical nature was developed until, in 1811, Blenkinsop obtained an English patent for the application of a rack laid all along the line of a railway, with a corresponding driving-pinion on the locomotive. This plan was at once executed on a railway leading from Leeds, in England to the Middleton Coal Pits. The opening of the line occurred August 12th, 1812, and though rack and engine were of an extremely primitive construction, yet this mode of operation remained in use for over twenty years in the transportation of coals from the Middleton Pits to Leeds. The pinion alone furnished the traction, and the engine worked on gradients of 1 in 15. Here, then, appears for the first time an effective means for operating heavy inclines by locomotive power. The action of the pinion in the rack is positive, and the amount of traction is limited only by the dimensions of rack and pinion, and is practically independent of the engine weight.

Fruitful as this idea would have seemed to be, and notwithstanding the fact that the light locomotives of the earlier railroad era excluded any economic operation of steeper gradients, the rack and pinion found no friends in its immediate adaptation to the regular railway system. The lines were universally developed with easy gradients—even Stephenson himself thought that the economical limit was reached with a gradient of 1 in 200, a limit which De Pambour later on extended to 1 in 100, and whenever a sudden rise or steeper gradients were encountered, they were overcome by means of ropes and stationary engines, much in the fashion that inclined planes in coal pits had already been worked for a long time. These inclined rope sections once formed a prominent feature on many an important railway in Europe, as well as in the United States; but one by one they were adapted to regular locomotive service, and with perhaps some few exceptions, on roads of secondary importance, they have long since become a mere matter of history. Inconvenience and expense have sealed their fates.

The so-called atmospheric railways formed a curious phase in the struggle for supremacy among the means of transportation. They took practical shape during the interval from 1840 to 1848, and occupied the closest attention of prominent engineers. Millions were spent in England and France in the perfection of these schemes, and millions

and schemes disappeared together. It had been expected that this mode of propulsion would be of special use in the operation of steep inclines, and it therefore deserves a passing mention.

In 1847, a bold attempt was made in our country to adapt the rack and pinion to the requirements of a regular railway line. The experiment took place on an inclined plane of 1 in 17 of the Madison and Indianapolis Railroad.* Besides the regular adhesion devices, the locomotive was provided with an independent and separately working steam engine for driving the pinion mechanism. Both adhesion and pinion worked simultaneously and assisted each other, and this co-operation constituted a signal advance over the original arrangement of Blenkinsop. But there was no end of repairs to the cast-iron rack, and these expensive repairs, and a somewhat defective design of the engines, led first to rope traction as a substitute, which, in the natural course of things, as elsewhere, gave way to locomotive service. As long as a cast-iron rack-rail had to be employed, this system had no possible chance for any larger applications, and with the state of the arts in those days, it was bound to fail. Everywhere adhesion was made to do the service as best it could.

Almost simultaneously with this described effort on the Madison and Indianapolis Railroad, George Escol Sellers, of Cincinnati, appeared with a plan for working inclines by laying a smooth center-rail, and by supplying the regular locomotive with a pair of horizontal friction rollers, pressing againt the center-rail, and being operated by a special set of cylinders. These friction rollers were to produce an additional tractive power sufficient to overcome very steep inclines. The proposition of a center-rail and horizontal friction wheels as a means to increase traction had already been made by Vignolles and Ericsson, previous to the historic competition on the Liverpool and Manchester Railway, but George Escol Sellers appears to have been the first one to distinctly propose the use of a smooth center-rail for climbing steep gradients. His plan, as laid down in a comprehensive and very lucid pamphlet, published in Cincinnati in 1849, contains many very ingenious suggestions and details, among which may be mentioned the clever manner by which

* A full account of the inclined plane near Indianapolis, with descriptions of the rack-rail, and of various locomotives and attachments used in connection with it, and also of the final use of the same plane by a gravity locomotive, is given in paper No. 158, by M.J. Becker, M. Am. Soc. C. E., in the Transactions of the American Society of Civil Engineers, Vol. VI, page 68 (March, 1878).

the resistance of the train was made to create the requisite pressure of the friction wheels against the center-rail. Mr. Sellers had a working model built, which was first exhibited in Cincinnati and subsequently in New York in 1848. When, stimulated by the all-exciting gold discoveries in California, the construction of the Panama Railway was decided upon, the first surveys led to gradients for which the ordinary means of locomotion appeared to be insufficient. This gave a chance for a practical demonstration of Mr. Sellers' plan on a grand scale, and the first engines for the Panama Railway were actually ordered and built upon his system. In the meantime, however, and before these locomotives arrived at the Isthmus, the present line, with comparatively light gradients, had been discovered. The horizontal adhesion mechanism became superfluous, and the engines were worked without it.

The year 1851 was one of special interest in the eventful development of the iron horse. It was the year in which the great competitive trial took place on the Semmering Railway in the Noric Alps of Austria, which was to decide on the best type of adhesion engine to work that difficult line, with its extensive gradients of 1 in 45, 1 in 42, 1 in 40, and with its numerous curves. None of the competing locomotives answered satisfactorily in the subsequent regular service, and this was productive for several years of many projects in locomotive construction for working steep gradients. Among these we notice the first appearance of the *double bogie engine* by Cockerill, in Seraing, Belgium, which, in very recent years, flashed up again into an ephemeral existence. We also recognize here Sellers' center-rail and mechanism in a proposition by Krauss, but all these plans found no favor, and had to give way to the *Engerth type* of locomotives, and its gradual transformation into the improved eight-couplers of our present day.

Once more we meet the center-rail of Sellers, when Fell, in 1867, applied it practically in his railway across the Mont Cenis. That line had an average gradient of 1 in 25.6, with maximum gradients of 1 in 12. It remained in actual operation until the tunnel was opened to traffic. This formed the last practical attempt with a smooth center-rail. The engine was rather complicated. The tractive power depended entirely upon the adhesion of smooth surfaces, and varied thus with the state of the weather and the conditions of the track.

But while Fell was busily engaged laying his center-rail across the Alps, Sylvester Marsh, in this country, was at work running a line with

a center rack-rail to the top of Mount Washington.* There could be no doubt as to the steepness of this road, its average gradient being about 1 in 4.1, with a maximum gradient of 1 in 2.67. The ground was first broken in the summer of 1866. With this Mount Washington Railway, the rack-rail system of Blenkinsop entered into a new era of development, and this was essentially due to the improvements in the rack itself. Cast-iron, with its low limit of strength and its unreliable qualities, was abandoned, and wrought-iron was used in its stead for the first time. The change in the material necessitated a change in the form. The rack took the shape of a ladder, the sides being formed by angle-irons and the rounds being the teeth or cogs, which were made of round iron. This railway was completely finished in 1869, and since then, up to the present hour, for a period of over 15 years, it has safely conveyed thousands of tourists to the lofty summit of Mount Washington without the loss of a life.

The first sections of the wrought-iron rack had hardly been laid at Mount Washington, when tidings of what Marsh was doing in America reached Switzerland, that world-famed mountain region. Shut in by, and at the same time separating, the leading commercial countries of the European continent, Switzerland had always been greatly benefited by the international traffic across its snow-capped mountain ranges. Keenly awake to this fact, she had always jealously watched her interests, and with the demands of growing trade, the bridle-paths and the mule caravans of a by-gone age gave way to magnificent road structures with the modern freight wagon and the stately post coach. The roads over the Splügen, the Bernhardin, the Lukmanier, the St. Gothard and the Simplon, are masterpieces in the art of road-making, and the boldness of some of their parts will always excite the wonder and the admiration of the traveler. But just then the public mind was in a state of growing excitement. The Brenner Pass in Austria, near to the eastern frontier of Switzerland, had just been opened to public traffic, and the whistle of the locomotive echoed and re-echoed merrily along the rocky line. Away down in the southwest, drilling machines were diligently at work in the heart of the Mont Cenis, making the big hole, where Fell had the audacity to climb the mountain even during the construction of

* See descriptions of Mount Washington Railway, by Charles W. Copeland, M. Am. Soc. C. E., and Francis Collingwood, M. Am. Soc. C. E., in the Proceedings of the American Society of Civil Engineers, Vol. III, page 12 (February, 1877).

the tunnel. These were threatening clouds for the vested rights of the little republic, and something had to be done to avert the danger. Projects of all sorts for working steep gradients were freely discussed, and it was a well-timed thing that the Swiss representative in Washington should send to his government an official report of what Marsh was quietly doing in the White Mountains. This happened somewhere in 1867, or the early part of 1868. It was clear that a railroad had to cross the Swiss Alps, and the St. Gothard Pass, on account of its central location, and notwithstanding its serious engineering problems, was the one most prominently discussed. In view of the prospective large traffic, and also of the shortcomings of regular adhesion service, it was not thought judicious to carry the gradients beyond 1 in 40, or thereabouts. But this was equivalent to an expensive development of the line and to a very long tunnel. It was in the autumn of 1868 that a proposition was made to adopt a ruling gradient of 1 in 20, and use the rack and pinion. This proposition came from N. Riggenbach, master mechanic of the Swiss Central Railway, conjointly with O. Zschokke, civil engineer, Though the adoption of the rack might have resulted in a saving of about one-third of the total cost of the road, yet the entire absence of practical experience on a line of this importance decided its rejection, and the St. Gothard was reserved for the adhesion locomotive.

The stone was rolling. The air was filled with enterprise, and if it would pay to run a railway up Mount Washington there would be tenfold less risk in doing the same thing on the Rigi, that most renowned and most frequented elevation of the world. In the spring of 1869 an engineer left for America to prepare a minute professional report on Marsh's enterprise. The necessary capital for the Rigi Railway was soon afterwards secured and operations were commenced. Thus Riggenbach introduced the wrought-iron "ladder-rail" in Europe in his Vitznau-Rigi Railroad, which was opened in 1871. All the details of this line are nothing but modified copies of the Mount Washington road. Engine and rack-rail—the principal dimensions being retained—were built stronger, more substantial, and with more care throughout. The round teeth, however, of the original ladder-rack were here abandoned in favor of a trapezoidal tooth and the system of involute gearing. The maximum gradients on the Vitznau-Rigi road are 1 in 4, and, like the Mount Washington Railway, it is essentially a pleasure line, serving to convey tourists up and down the mountain sides.

About the time that work was commenced on the Rigi Railway, Riggenbach started also to build a short line connecting the sandstone quarries of Ostermundingen (Berne) with an already existing adhesion line. The same wrought-iron ladder-rack was employed as on the Rigi enterprise, but in other respects there were essential differences. On the Rigi the rack-rail is laid throughout the entire length of the line; here, however, we find the line made up of rack-rail and of pure adhesion sections, the rack-rail being only used where the gradients required large traction. The maximum gradient is 1 in 10. The locomotives were of such a construction that either the pinion or the adhesion drivers could be thrown in or out of connection with the steam cylinders, and the train could thus be either propelled by adhesion alone, or by the pinion and rack alone, according to the gradients of the road. This line has remained in successful operation to the present day, and, excepting the short-lived attempt on the Madison and Indianapolis Railroad, we have here the first instance of a practical combination of the rack-rail with pure adhesion portions.

The immense financial success of the Rigi Railway, together with the fact that thousands upon thousands from all parts of the civilized world assisted every year to spread the fame of that remarkable spectacle of the puffing locomotive on the mountain side, soon led to the construction of other rack-railroads. The Mount Washington road had given the first impulse, the Rigi did the advertising, and the line to the quarries of Ostermundingen pointed the way to every-day applications. The total aggregate of the lengths of rack-railways laid in Europe alone up to the present time is about 28 miles, but it must be remembered that the Mount Washington line, which only has a length of 2.8 miles, if built with a uniform gradient of 1 in 40 would be over 27 miles in length, and would have to be run at 29 miles an hour to be the equivalent in time of the Mount Washington Railway at 3 miles an hour. The following is an enumeration of the lines thus far built, all of them being still in regular operation.

RAILWAYS WITH RACK-RAIL LAID OVER THE ENTIRE LINE.

	CONSTRUCTED.	MAX. GRADIENT.		RADIUS OF CURVES.
Mt. Washington, New Hampshire	1866-69	1 in 2.67	37.5 per cent.	Feet. 554
Vitznau-Rigi, Switzerland	1870	1 in 4	25 "	590
Kahlenberg-Vienna, Austria	1872	1 in 10	10 "	590
Schwabenberg-Pesth, Hungary	1872	1 in 10	10 "	590
Arth-Rigi, Switzerland	1874	1 in 4.8	20.8 "	590
Rio de Janeiro, Brazil	1882	1 in 6.7	15 "	
Drachenfels to the Rhine, Germany	1883	1 in 4.5	22.2 "	

RAILWAYS WITH RACK-RAIL AND ADHESION SECTIONS COMBINED.

	CONSTRUCTED.	MAX. GRADIENT.		RADIUS OF CURVES.
Ostermundingen, Switzerland	1870	1 in 10	10 per cent.	Feet.
Rorschach-Heiden, "	1874	1 in 11	9.1 "	790
Wasseralfingen, Germany	1876	1 in 12.5	8 "	395
Rüti, Switzerland	1877	1 in 10	10 "	345
Laufen (Quarries), Berne, Switzerland	1878	1 in 16.7	6 "	345
Oberlahnstein, Germany	1880	1 in 10	10 "	

The railways with rack-rail and adhesion sections combined, as a rule are worked in the manner described with the Ostermundingen line, that is, the engine exerts traction either by adhesion or by the pinion alone; yet in several instances attempts have been made to utilize adhesion simultaneously in combination with the pinion by simply coupling the adhesion drivers with the pinion, and notwithstanding the grave practical objections which stand in the way of such an arrangement, locomotives of this type can still be found.

These lines gradually and unmistakably demonstrated not only the feasibility, but the great practical bearing of the rack-rail for working steep gradients. They showed that inclines could be safely operated, which with any failure in the mechanism would of necessity lead to terrible catastrophes; but so reliable were the provisions against such an event, that the first accident has yet to be recorded. They also showed that sharp curves could be rounded with ease, and, irrespective of the

state of the weather, or of the conditions of the track, the maximum train weight could always be hauled over the line.

In the meantime work had been progressing on the long tunnel through the St. Gothard. The cost was greater than had been anticipated; and in 1876, when work was to be commenced on the approaches to either side of the tunnel, the company found itself in very straightened circumstances. Once more the application of the rack-rail and the adoption of steeper gradients was discussed, and this time with great seriousness. It was expected to finish the road with the means at hand. A plan was worked out. The project was warmly supported by several prominent continental engineers; but, though the feasibility of the system was acceded to in principle on all sides, no definite conclusion could be arrived at, and the subject was ultimately tabled on account of the resistance of the Italian delegates, and a somewhat general reluctance to adopt the system in its then undeveloped state.

Defects of the Ladder Rack-Rail.

Following thus the historic developments of the means for working steep gradients, we have been able to see how, little by little, the rack-rail has established for itself a certain modern recognition—a recognition, too, which is founded upon stubborn facts furnished by experience. There can no longer be any doubt concerning the practicability of the rack as a mechanical device in all these applications; and wherever once introduced in its modern form it has kept its own ground, thus furnishing a proof of the economy of its use. Yet withal it must be recalled that the existing rack-railroads are of moderate extension, of moderate traffic, and are operated at moderate velocities only. When it really comes to the higher demands of a regular railroad service, we find that the ladder-rack of Sylvester Marsh has serious inherent defects which incapacitate it for such purposes, and for which defects the very form of the rack excludes all remedies. The prime requisite for a smooth and quiet motion in all gearing is uniformity of pitch; and the greater the velocities the more important is this uniformity. It is the recognition of this fact which has led to the elaborate gear-cutting and gear-milling tools of our days. But the very mode of manufacturing the ladder-rack, the punching of its sides, the riveting of teeth and sides, etc., introduces elements of error which, though small by direct measurement become, apparent at higher speeds. Then, also,

the ladder-rack is built in sections of about 10 feet, so as to reduce the influence of expansion by temperature. Still that influence is there, and it exists at every joint in form of an unequal pitch. The velocity on the existing rack-railroads, and on the steeper gradients, is generally from 3¾ to 4¼ miles per hour. At 5 miles per hour, the operation of the pinion in the rack much resembles a sort of trip-hammer action; and every increase of speed tells in the rapid wear of the pinion, though made of the best of steel. With an inability to run at high speeds, the locomotive, though running slow on steep gradients, and working with its full boiler capacity, cannot run correspondingly faster on the rack-rail sections of easier gradients where the traction is less. The engine, therefore, is unable in such cases to exert its full power, and this involves loss of time and an increased cost of transportation.

In order that the motion of the pinion in the rack—even under the supposition of an absolutely uniform pitch—be free of blows or impacts, it is necessary that before one pair of teeth has gone out of contact, another pair must have come to fairly bear against each other. This important condition determines the size, *i. e.*, diameter, of the pinion for a given pitch. The size of the pitch again depends upon the kind of materials employed, the amount of traction, and the width of the teeth. In the present ladder-rack the pitch is already as small as it possibly could be chosen, and still the necessary diameter of the pinion becomes so large, that, together with the low speed, the power from the steam cylinders has to be transmitted to the pinion by multiple gearing. But multiple gearing complicates the locomotive considerably; it introduces additional frictional losses; its renewal from time to time is expensive; and the additional rotating masses between the pinion and the elastic cushioning steam are not conducive to a soft action.

In the present ladder-rack the strains are also very high—especially so on the curves; in fact they are greater than what generally would be considered a safe working strain. A radical increase of traction, as the heavy traffic of a first-class regular line might demand, is altogether beyond the capacity of the ladder type. Considering the inequalities of pitch due to manufacture, and still more the discontinuity of the rack, its 10-feet sections, and the chance of slight derangements, it becomes evident that an increase of traction by means of the direct coupling of two driving pinions is a thing utterly impossible. Any strengthening of the ladder-rack by an increase in the size of the pitch would only

be creating most serious difficulties of another kind, recollecting what has been said about multiple gearing.

The ladder-rack has done well enough in its present sphere and under its present conditions, though even here suffering from its constitutional troubles; but when the imperious demands of an ever-progressing age are to be satisfied, then the ladder principle sadly fails. Its defects are inherent; they are of the blood and flesh of its form; and to free the sound and healthy rack principle from those drawbacks, and expand its usefulness, required the discovery of an entirely new type. This, we venture to say, has been admirably accomplished by Mr. Roman Abt. Mr. Abt has been for many years chief constructing engineer in Riggenbach's various rack enterprises. Many of the applications of the rack-rail in Europe were made under his immediate care, and, if any one has been able to thoroughly study and master the subject, he has. His new rack system embodies the experience of years in this class of engineering, and it is the writer's opinion that the rack, which thus far had been regarded more as a sort of makeshift, to be applied in extraordinary cases which required extraordinary means, has been advanced by him into a most important element in the planning of new roads, or in the economical operation of old ones.

Abt's Rack-Rail. (See Plate V.)

In the construction of his new rack-rail, Mr. Abt has entirely abandoned the hopeless ladder-type, with its two sides and its rounds or teeth in between. His rack-rail consists essentially and is built up of a number of individual elementary racks placed side by side. Each one of these elementary parts is nothing but a rack in the very simplest form, which is that of a bar of given length provided with teeth. Such an elementary rack will be able to stand a certain safe working strain, which is positively determined by the material, the dimensions of the bar and the pitch of the teeth. Whatever may be the traction to be developed by the pinion, by placing side by side a sufficient number of elementary bars, each one strained only to its chosen limit of working strain, we are able to meet any amount of traction as far as the strength of the rack-rail is concerned. These bars are connected among themselves by iron or steel chairs and bolts. The chairs, which are placed at regular intervals, also keep the bars laterally distanced from each other, and through these chairs the whole is firmly bolted or otherwise secured to the wooden or iron ties of the road-bed.

PLATE V.
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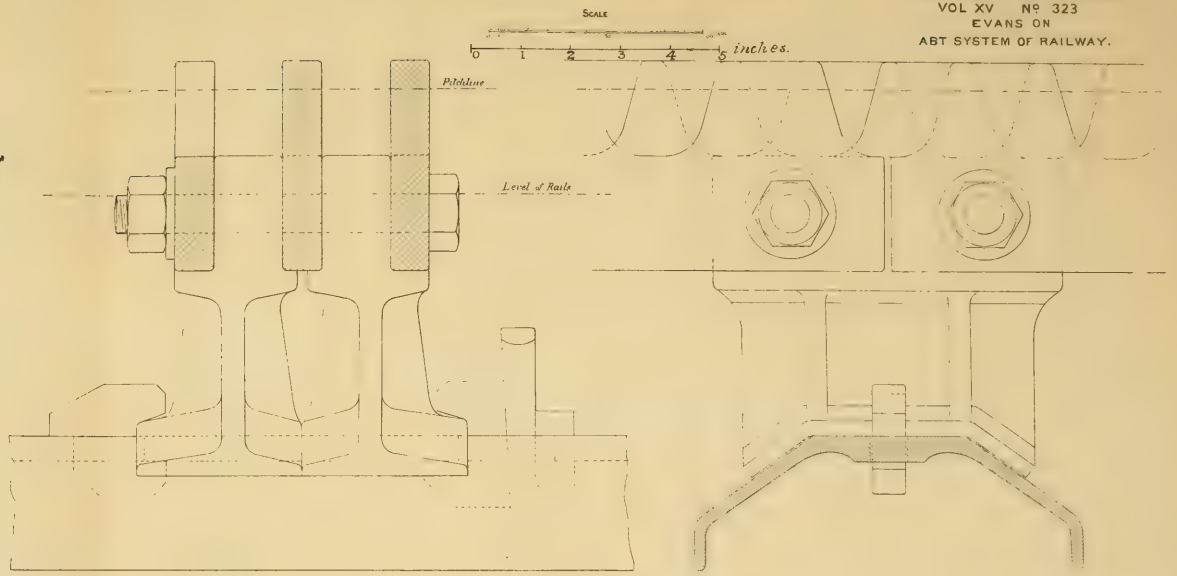
EVANS ON
ABT SYSTEM OF RAILWAY.

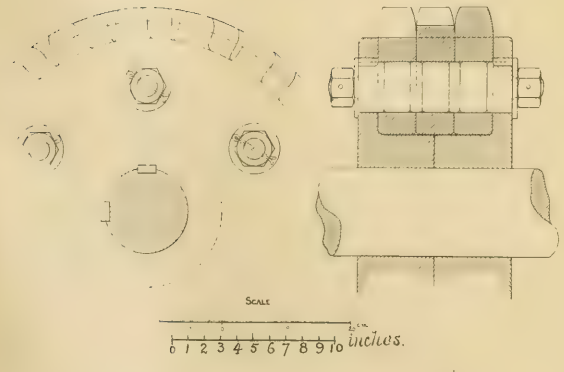
THE ABT SYSTEM OF RAILWAY

(Harz - Railway)

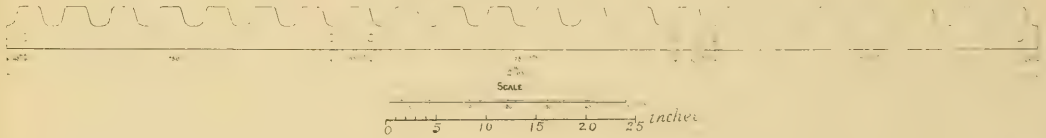
RACK RAIL & CHAIR

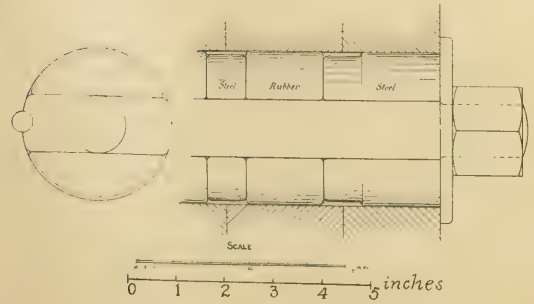


PINION

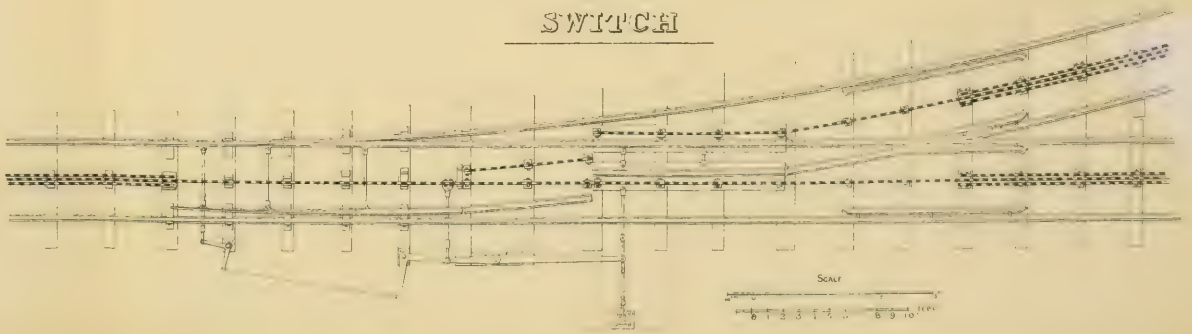


RACK BAR





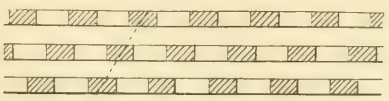
SWITCH



The material of the bars may be either rolled-iron, Bessemer steel, cast-steel, or eventually also a good quality of steel castings, and the choice will be governed by convenience, or by the requirements of the road. With rolled bars the teeth may be produced in manifold ways, be it by punching, drilling, milling, sawing, pressure in dies, etc.; but no matter how produced, the last finishing touch ought to be always given by milling, which secures a perfect uniformity of pitch, the one important point to be aimed at. It guarantees a perfect contact between rack and pinion, and with the facilities of the modern machine-shop its expense is no barrier to its application.

Mr. Abt lays the various bars, of which his rack-rail is composed, with broken joints. In this manner his rack-rail becomes practically one continuous structure, always one or more whole bars passing the joint of every two bars. Through this continuity of the rack-rail, and through the fastenings to the ties, a most desirable and important union is established between these two elements of the road, and still old ties may be exchanged for new ones without causing the slightest difficulty in the maintenance of the line. The breaking of the joints has the further advantage of equalizing throughout the influence of expansion by temperature. Comparing this with the ladder-type, with its discontinuous 10-feet sections, the advantages of Abt's rail with its continuous solidity and strength are very striking.

But the component bars of Abt's rack-rail are not only laid with broken joints, they are also, if the expression be permitted, laid with broken pitches—that is, the teeth of the side-by-side bars are arranged in the form of steps, as shown below, which is a beautiful feature of



Abt's rail. It enables the simultaneous contact of several teeth of the pinion with different bars of the rack-rail, thus giving a contact and a smoothness of motion that could never be obtained with the ladder-type. This interlocking of several teeth offers at the same time an increased security against the fracture of a tooth, and even if, from some external cause, such an event should occur, it could not be followed by any serious consequences. In discussing the defects of the ladder-rack, we have seen that for the very reason of getting just

sufficient contact, the pinion had to be made of a certain size, and with that size multiple gearing became unavoidable; through the stepping of the teeth, however, Abt's pinion can be made so much smaller in diameter, without interfering with a still superior contact, that multiple gearing is entirely unnecessary, and is dispensed with. The ensuing simplicity and efficiency of the locomotive is not the least of the many excellent points of Abt's system.

The pinion is constructed of as many separate disks as there are separate bars in the rack-rail, each disk corresponding to a line of bars, and the disks are shifted against each other according to the stepping of the teeth in the bars. Another very important feature in the construction of the pinion, is that its various disks are in an elastic connection with the shaft by means of inserted springs. (Plate V.) By this arrangement the different disks obtain a small, limited, relative motion to each other. We had occasion to point out the sources of error which would lead to imperfections in the manufacture of the ladder-rack; the manufacture of Abt's rail, on the contrary, admits at every step of the application of tools which secure accurate work. Yet whatever remains of slight inaccuracies, be it in manufacture, be it due to the mode of fastening on the ordinary ties, or be it due to expansion by temperature, they are most cleverly counteracted by an elastic connection of the pinion disks with the shaft; and an absolutely perfect, automatic contact of the teeth is established, with the pinion traction uniformly distributed over the various bars of the rack-rail. Practical experience has fully borne out the high expectations that were entertained in the conception of this system. Even at 16 miles per hour a motion is obtained on rack-rail sections which is perfectly smooth and free from any blows or irregularities. Not the slightest noise is to be observed, the pinion working as silently in the rack as the regular adhesion drivers on the ordinary rails.

A characteristic property of Abt's rack-rail consists in its economic accommodation to the gradients of a road. The numbers or the thickness of the individual bars of which the rack-rail is composed, are chosen according to the traction to be sustained; number or thickness of the bars, therefore, increase and decrease with the gradient. Such an accommodation is impossible with the ladder-type for obvious reasons.

Then, again, curves of any radius can be laid with ease, because the same bars are used as on the straight portions of the line. The ladder-

rack, on the contrary, has to be made on purpose, with great care, and for each radius separately.

On the Mount Washington, Rigi and several others of the earlier railroads, the rack-rail rests on wooden stringers, which in turn are fastened to the ties. With the introduction of the mixed, or combined adhesion and rack-rail, systems it became necessary to give to the rack-rail a sufficiently elevated position relative to the ordinary rails, so that the pinion could clear the latter safely at switches and crossings. Iron chairs were now introduced for support. The free position of the rack-rail obtained in this manner is an excellent safeguard against accumulations of snow and mud. With Mr. Abt's system this advantage exists in a higher measure, on account of the narrow bars of the rack-rail and the comparatively wide clearances between them. These clearances form a kind of furrow, which admit of a mechanical cleaning, or, we might say, plowing, by a device on the locomotive itself. As mountain regions will particularly be the haunts of rack and pinion, and mountain regions generally have heavier snow-falls, it will be of interest to cite the experience of the existing lines. Ostermundingen and Rorschach-Heiden, in Switzerland, and Wasseralfingen-Friedrichsberg, in Germany, are all three operated summer and winter, and at all these places severe winters are prevalent. The lines were constructed respectively in 1870, 1874 and 1876; they were opened to traffic in the years immediately succeeding, and have been regularly operated ever since. Yet in all this time trains have run on these lines in the heaviest snow-storms, and not a single train has ever been stopped on account of snow or snow-drifts. The rack-rail locomotive is provided with a sort of snow-plow, and the positive action of the pinion always pulls it through. The Fourth Annual Report of the Rorschach-Heiden road says: "The extremely heavy snow-falls of last year not only did not influence the regular operations of the line, but we had no expense whatever for removing the snow. Experience shows that a heavy snow is far less obstructing to a rack railroad than to an adhesion line; no matter how large the accummulated masses are, with a rack railroad they are of no account." Practice has demonstrated that the rack locomotive can pass snow-drifts which the adhesion locomotive could not overcome. The rack-rail on the above lines belongs of course to the ladder-type, but no difficulties with snow having appeared there after so many years of constant service, none will exist with the superior Abt rack-rail.

Crossings and switches are made and operated with rack-railroads similar to those on pure adhesion lines, and in every respect they are as simple and as safe. The first switches were constructed in the form of sliding-tables and turn-tables, as, for instance, on the Mount Washington, and immediately afterwards on the Rigi Railway, but they soon were built in close adherence to the customary adhesion devices. (See Plate V.)

The peculiar method by which, on a line of the combined system, the change is made from the adhesion portions to the rack-rail without stopping the train, will be described after having discussed Abt's locomotives.

Abt's Locomotives.

The rack-rail enables us to adopt steeper gradients in the development of our railroads; we can follow more closely the natural slopes and shorten our line. But by combining with the steeper gradients the easier ones of the pure adhesion system, we expand the usefulness of the rack-rail enormously, provided we are also capable of building locomotives which, under all the varying conditions of grade, can invariably exert their full power. This means that traction and corresponding speed must vary in inverse proportion: large traction with low speed, small traction with high speed. If we simply take a line with a maximum gradient of 6 per cent., and with other gradients ranging all the way down to level, the condition of a full exhaustion of the engine's power in every part of the line demands a most extraordinary capacity of accommodation. On adhesion portions an adhesion engine will do the work, and will do it well if properly dimensioned, but once those gradients are transgressed which practical experience has designated as a limit to an economical adhesion service, the locomotive necessarily must fail, not only on account of lack of traction, but also, and which is equally important, on account of lack of purchase for the steam. Weight, that recourse of regular locomotive practice, together with increased cylinder volume, does not offer any satisfactory solution, for while it slightly improves the climbing qualities, it introduces drawbacks in its stead by which the benefit derived has to be dearly paid for all over the line. With the rack and pinion, on the other hand, weight is practically immaterial for their working. We could therefore take any type of adhesion locomotive which, with comparatively moderate weight, has been approved of for practical adhesion gradients, and by adding the pinion

THE

ABT SYSTEM OF RAILWAY

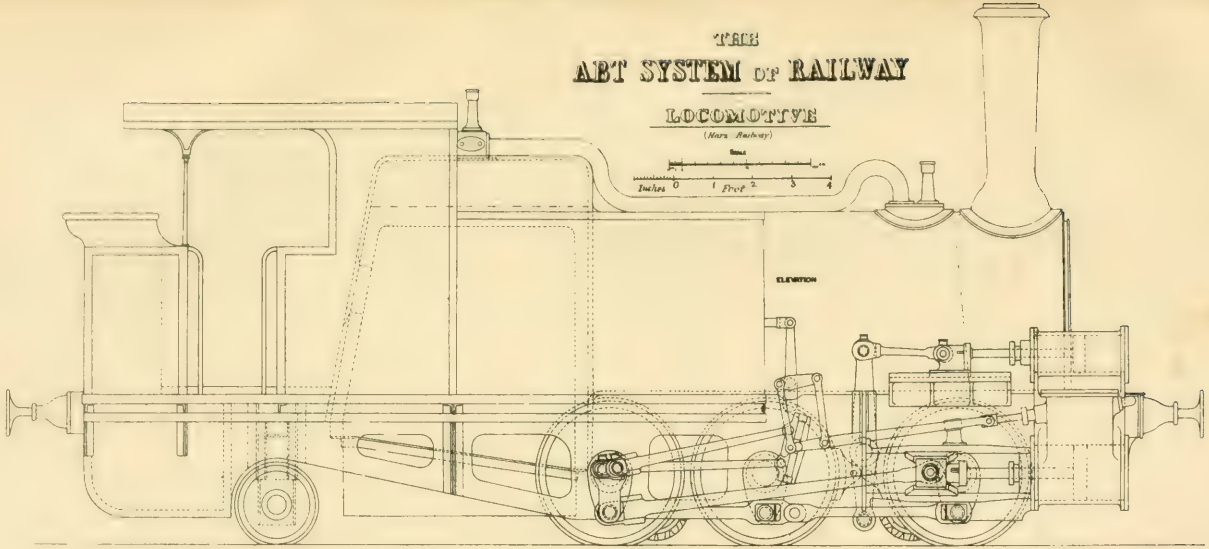
LOCOMOTIVE

(Mars Railway)

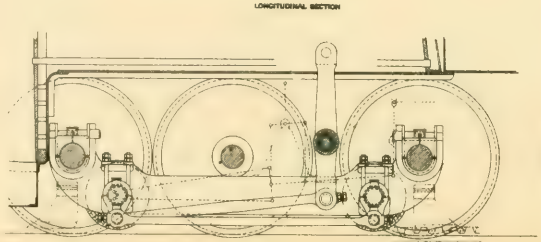
Scale

Inches 0 1 2 3 4

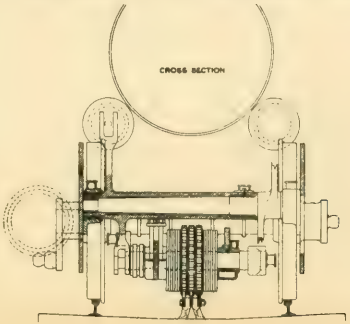
Feet



ELEVATION



LONGITUDINAL SECTION



CROSS SECTION

PLATE VI.
TRANS. AM. SOC. CIV. ENG'RS.
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EVANS ON
ABT SYSTEM OF RAILWAY.

mechanism as an independent and separate part throughout—with an independent set of steam cylinders—we could add exactly those qualities which are indispensable in the mounting of the steeper gradients. This is precisely what Mr. Abt has done in designing his locomotives. Adhesion drivers work separately and independently from the pinion device; they work alone on the pure adhesion portions of the line, and they quietly keep on working when rack-rail sections are encountered. Rack and pinion then supply the additional traction, and the extra steam cylinders secure the necessary purchase for the steam. The speed adjusts itself according to the total traction and the steaming qualities of the boiler, and the engine develops its full powers under the most varying and trying of circumstances.

To the exhausts of the ordinary adhesion cylinders are added, when ascending rack-rail portions of the line, the more numerous exhausts of the pinion cylinders. This produces an energetic and uniform blast, and secures a free and abundant steaming at the very time it is most needed.

Another important reason for utilizing the natural adhesion of the locomotive, exists in the fact that, even on gradients of 600 and more feet to the mile, and with trains of a still practical size, the traction due to adhesion alone will be from $\frac{1}{2}$ to $\frac{1}{3}$ of the total traction. The rack-rail can thus be relieved of a corresponding amount of strain, and can be made lighter and cheaper. A certain engine weight we shall always have, for weight and the capacity of performing work are, to a certain extent, parallel terms in locomotive construction, and this necessary weight can be excellently made use of in relieving the strain on the rack.

The characteristic feature, then, of Abt's locomotives, consists in the additional mechanism with which any well approved or favored type of adhesion engine is equipped, and thus is transformed into a locomotive for a rack-railroad. Through Mr. Abt's ingenious rack-rail construction, which admits a pinion of small diameter, multiple gearing, as we stated previously, has become superfluous; the pinion shaft is driven directly from the separate pair of cylinders, and the construction of the rack-rail locomotive is reduced to a simple form.

The vast superiority of the independent adhesion drivers and pinion over their combination by direct coupling, as has been done on some of the modern rack lines, is evident. A rigid coupling of the two mechanisms presupposes an ever-constant relationship between the pitch-line

diameter of the pinion and the diameter of the adhesion drivers, a condition of things which practically it is impossible to maintain. Important frictional losses are unavoidable with such a design; then, besides, there being only one pair of cylinders—unless there should be a special second train of wheel-work—the engine lacks that capacity of economical accommodation which forms such an important feature of Abt's locomotive. In this engine of his, the separation of the adhesion and the pinion device is so absolutely perfect that neither one can disturb the other. Whether on account of the condition of the track the adhesion drivers slide or not, whether they be running ahead or behind, all this has not the slightest influence on the pinion, which steadily does its part.

It is true that on the engines of the Madison and Indianapolis Railroad the pinion was driven by independent steam cylinders, but Abt's engine is otherwise entirely different in construction. Then, again, Abt's whole railway system is essentially grounded upon his rack-rail, and his engine is the logical sequence of the latter.

Through Abt's rack-rail it has become possible to build rack-rail locomotives of special large traction, *by simply coupling two or more driving pinions.* All the objections which existed with the ladder-rack against the coupling of pinions as a means to increase tractive power, are completely overcome in Abt's rack-rail type. With its continuity, its perfect and equalized pitch, and the elasticity in the pinion disks, coupling has not only become practicable, but by placing one pinion a little in advance of the other in reference to time of contact, coupling only heightens the uniformity and smoothness of motion.

In his original engine design, Mr. Abt selected the Engerth type of adhesion locomotive. There were six coupled adhesion drivers. The tender carried the pinion shaft proper with the one driving pinion in the center. At either end of the pinion shaft were brake disks, to which the crank pins were attached. The tender frame supported the extra steam cylinders, and steam was conducted to and fro through a specially designed ball and socket coupling of locomotive and tender.

In his Harz locomotives (Plate VI; see also page 177), the first ones actually built upon his system, Mr. Abt adopted the adhesion type of a six-coupled tank engine, with a trailing axle behind the fire-box. They have two coupled driving pinions placed under the front part of the locomotive, and the additional pair of cylinders is arranged above

those for the adhesion drivers. The two pinion shafts rest in a frame hung to the first and third of the adhesion axles. Motion is transmitted from the cylinders above to the pinions below by means of rockers, in a very compact and clever manner. The size of the cylinders is small, as the piston velocity, by reason of the small pinions and the interjacent connections, is large. The frame of the pinion shafts, being suspended directly to the adhesion axles without any intervening springs, establishes the proper relative position of pinion and rack. The constant relative position between the rack-rail and the ordinary rails is preserved through the regular maintenance of the line, which experience has demonstrated is easily done. Another protection against any incorrect contact between the teeth, on account of a variation in the depth of interlocking between pinion and rack, exists in the system of involute gearing itself. This system possesses the valuable quality of the contact remaining always correct, though the depth of interlocking may undergo variations. The natural wear of the adhesion tyres is compensated from time to time by a readjustment of the bearings.

The link gear is, of course, separate for each of the two sets of steam cylinders. For the adhesion cylinders the starting levers or screws are arranged in the usual manner, with a horizontal motion, whilst these levers or screws for the incidentally working pinion cylinders are given a vertical motion; thus well discriminating between the two.

The braking arrangements are very efficient. There is first an ordinary brake for the adhesion drivers; then there are brake disks on the crank shaft of the pinion mechanism, by means of which the pinions may be completely locked; and thirdly, there is an auxiliary loose pinion, with brake disks, on the trailing axle of the locomotive. The last one serves merely for the purpose of stopping the train in case of any accident to the driving pinions or their gear. The braking on down gradients is done by using the steam cylinders as air-compressors, there being special provisions for this purpose. This is the most perfect brake imaginable, and it is used on every one of the existing rack-railroads. In regular service all down-hill braking is entirely done by the driving pinions of the locomotive, and whatever braking pinions in the train are being called upon—one or more cars being provided with such. Wherever, therefore, the rack-rail is laid, the wear and tear of braking is taken off from the rails and tyres without proportionately diminishing the lives of rack-rail and pinion, because the sliding and rolling of the

teeth in contact occurs on smooth and well-greased surfaces. The work of braking is chiefly spent here in the compression of air in the steam cylinders, while with the usual braking device, *i. e.*, brake shoes pressing against the wheels, it is busily engaged in the demolition of rails, tyres and shoes.

Mr. Abt, when designing the normal type of his engines, which were to run on lines connecting directly with existing adhesion railways and using the regular rolling stock, had to limit the net tractive power to the working capacities of the draw-bars and the buffers in use. The normal draw-bar of the European lines possesses a working pull of 14 300 to 15 400 pounds, while the normal buffers admit a working compression from 16 500 to 17 600 pounds. Mr. Abt, in order to get as powerful an engine as possible, chose the latter figure of 17 600 pounds, and the locomotive when exerting the maximum net traction has its place at the rear of the train, and pushes it ahead. Besides an increased safety in the operation of lines with heavy gradients, there are still other reasons, which we will touch further on, why the engine is preferably placed to the rear.

The 17 600 pounds then represent the net traction of the locomotive; whatever traction is necessary to move the engine itself has to be added in order to obtain the gross traction; that is, the combined effort of pinion and adhesion drivers. It is evident that for lines with special rolling stock, or with cars having extra strong draw-bars, locomotives of far greater tractive power can be built.

The conditions for a powerful locomotive of this class are more favorable in our country, because the draw-bars are substantially stronger. Our draw-bars are capable of standing a safe working pull or push of at least 21 200 pounds, and we may adopt this figure for the present as the net traction of the normal American types of Abt's engines. These engines will weigh 50 gross tons. The following traction table gives the net train weights in gross tons, exclusive of locomotive, which these engines will haul on the various gradients; it also gives the corresponding speeds in miles per hour.

TRACTION TABLE A.

GRADIENTS.			TRAIN WEIGHT.	SPEED.
Per Cent.	Ratio per Unit of Rise.	Feet per Mile.	Gross Tons.	Miles per Hour.
3.	1 in $33\frac{1}{3}$	158.4	290	8.6
3.5	1 " 28.6	184.8	250	8.4
4.	1 " 25.	211.2	222	8.2
5.	1 " 20.	264.	180	7.8
6.	1 " $16\frac{2}{3}$	316.8	152	7.5
7.	1 " 14.3	369.6	130	7.2
8.	1 " 12.5	422.4	115	6.9
9.	1 " 11.1	475.2	102	6.6
10	1 " 10.	528.	92	6.4
11.	1 " 9.1	580.8	84	6.2
12.	1 " $8\frac{1}{3}$	633.6	77	6.
13.	1 " 7.7	686.4	71	5.8
14.	1 " 7.1	739.2	66	5.7
15.	1 " $6\frac{2}{3}$	792.	62	5.5

On pure adhesion sections the speed of these engines may be run up to 30 and 35 miles per hour, while on rack-rail sections of lighter gradients a velocity of 12 miles per hour can safely be maintained in regular service. And such a speed we may need even on rack-rail sections if the system is to answer all the possible requirements of a regular traffic in passengers and merchandise.

When comparing our traction table with existing tests of adhesion engines, it is to be remembered that the table is based upon a net pull or push at the draw-bar of 21 200 pounds. Any larger admissible draw-bar strain will enable us to build our engines with proportionately greater traction, so that they will haul heavier trains. Another point worthy of being considered is, that our table does not present holiday dresses nor parade performances with exceptional conditions of track and sand-box assistance, but the figures can be relied upon as the regular every-day train-weights, irrespective of the weather or track. When making this statement we have not lost sight of the functions of the adhesion drivers, and we may add in explanation that their apportioned traction is based upon a low coefficient of adhesion. When the engine performs its maximum duty, then the steam is working at half-stroke admission.

The enormous adhesion engines which have been built of late, make

very severe demands on the capacities of the draw bars and the strength of the cars, in some instances the draw-bar strain rising to 27 500 pounds. We thought it might be instructive to add a second Traction Table, B, based upon a net traction of 27 500 pounds, the engines weighing again 50 gross tons.

Traction Table B.

Per cent.	Gradients. Ratio per Unit of Rise.	Feet per Mile.	Train Weight. Gross Tons.	Speed. Miles per Hour.
3.	1 in 33⅓	158.4	410	6.4
3.5	1 " 28.6	184.8	355	6.3
4.	1 " 25.	211.2	314	6.2
5.	1 " 20.	264.	254	6.
6.	1 " 16⅔	316.8	213	5.8
7.	1 " 14.3	369.6	184	5.7
8.	1 " 12.5	422.4	162	5.5
9.	1 " 11.1	475.2	144	5.4
10.	1 " 10.	528.	130	5.2
11.	1 " 9.1	580.8	119	5.
12.	1 " 8.⅓	633.6	109	4.9
13.	1 " 7.7	686.4	101	4.8
14	1 " 7.1	739.2	94	4.7
15.	1 " 6⅔	792.	87	4.6

To build the engines for such a traction is easy enough, if only draw-bars and cars will stand the pull or push of 27 500 pounds.

It is self-evident that engines of smaller weight and power can be constructed to suit any requirements of traffic. For roads of light traffic and smaller trains, a special engine has been designed by Mr. Abt, with only one pair of steam cylinders for both adhesion drivers and pinion. In this case, of course, only one device can work at the same time. This locomotive resembles, in its action, the engines on some of the existing rack-railroads of the combined system, but the whole construction is by far simpler and better.

A very important question relates to engine repairs. Experience has taught that, even with the complicated engines with multiple gearing, the repairs are perfectly normal, and can be compared favorably to those of regular adhesion locomotives when doing the same amount of work.

Automatic Entering Rail. (See Plate VII.)

On a line of the combined system it is of the greatest moment that the rack-rail be properly entered by the pinion without slackening the speed of the train, especially if the line happens to consist of a great many alternating sections. Mr. Abt accomplishes this by laying down a special entering rack-rail. This entering rail, of about 10 feet in length, is hinged upon the end which joins the continuation of the rack, and its other free end rests upon strong spiral springs. The teeth toward the free end gradually diminish in height, giving that end a wedge-shaped appearance when looking at it from the side. When the locomotive passes over the entering rail, steam is not yet admitted to the extra pair of cylinders, so that the pinion upon touching the yielding entering rail, and on account of the friction produced thereby, can turn about its axis, and within a few feet of distance traveled over, a true interlocking of the teeth is reached. Should the friction between the pinion and the entering rail be too small to produce a rotation of the pinion, then the latter will slide along the entering rail, and after a few inches of sliding the proper contact is secured. The very simplicity of this apparatus is the best guarantee of the absolute certainty of its automatic operation. It never has failed, and it works without any blows or noise.

When reflecting upon the working of the locomotive all along the line, apprehensions might be felt at first glance on account of the apparent complexity of the parts, and there being but one driver to take care of and manipulate the whole. But it must be considered first, that the locomotive is almost invariably placed in the rear of the train, and, secondly, that the various manipulations come about at different times and never simultaneously. With regard to the first point, one of the most trying and exhausting of the duties of the driver on a regular adhesion locomotive consists in the watching of the line with all its flying pictures, crossings, curves, signals, etc., and particularly so when the train is a fast express. His attention is divided between engine and line, and the greater part is absorbed by the latter. With the engine at the rear of the train, the line is watched by the brakeman on the front car, and the driver, thus greatly relieved, has mainly to care for his engine. On the adhesion portions of the line he has nothing but the adhesion parts to look after. When approaching a rack-rail section the speed of the train is moderate, the entering rail being passed at a speed of 5 to 6 miles

per hour, and he has ample time to watch the correct entering of the pinion before he gives steam to the extra set of cylinders. His adhesion engine needs no attention whatever at that moment, but is working steadily along. On down-hill gradients again all he has to care for are his brakes. There is only one thing to do at a time and plenty of time to do it well.

Having described the essential features of Abt's rack-rail and engines, it remains yet to say something of the practical applications of his system.

Applications of the System.

The location of railroads in mountainous districts has ever been one of the most difficult tasks of the engineer. Comprehensive knowledge, ingenuity and skill have planned many a wonderful line over mountain ranges and ridges, but the victories invariably had to be dearly paid for, and more than one of them has been a defeat for the unfortunate shareholder. Hemmed in by narrow precipitous valleys, winding their sinuous course around projecting mountain sides, sometimes abruptly changing their average gradients in steep ascents, the question of cost of construction is intimately associated with the limit of gradient which the prospective traffic may warrant for an economical management. Here each special case will present peculiarities of its own, but for important trunk lines or main arteries, like those that cross the Alpine chains of Europe, experience, running clear down to the present days, proclaims a gradient of 1 in 40, or $2\frac{1}{2}$ per cent., as the one yet compatible with an economic and safe management of a pure adhesion road. The Semmering and the Brenner are both built with this ruling gradient. At the Mont Cenis, to be sure, we find a 3 per cent. gradient for a considerable portion of the line, yet on the St. Gothard Railway, soon afterward, it was not thought judicious to go beyond $2\frac{6}{10}$ per cent. or 1 in $38\frac{1}{2}$, and most of the line follows 1 in 40. Higher gradients not only make severe demands on the tractive power of the locomotive, but, especially in mountain regions, where local dampness and frequent moist deposits materially reduce adhesion, they threaten also a safe and regular operation of the line, and this is one of the gravest objections against the application of steeper gradients for adhesion service. The lower the limit of gradient, the more serious will be the conquering of a rapidly ascending valley, or a sudden rise to some table land above. Instead of being able to follow the cheaper course along the water lines or valley bottoms, we

THE ABT SYSTEM OF RAILWAY

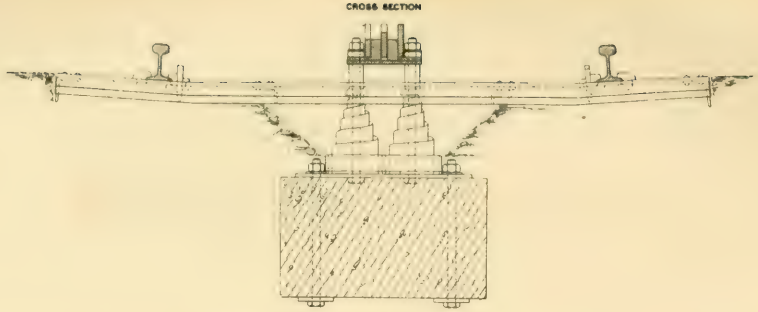
ENTERING RAIL

(Harz - Railway)

SCALE

0 5 10 15 20 25 inches

CROSS SECTION



ELEVATION

PLAN

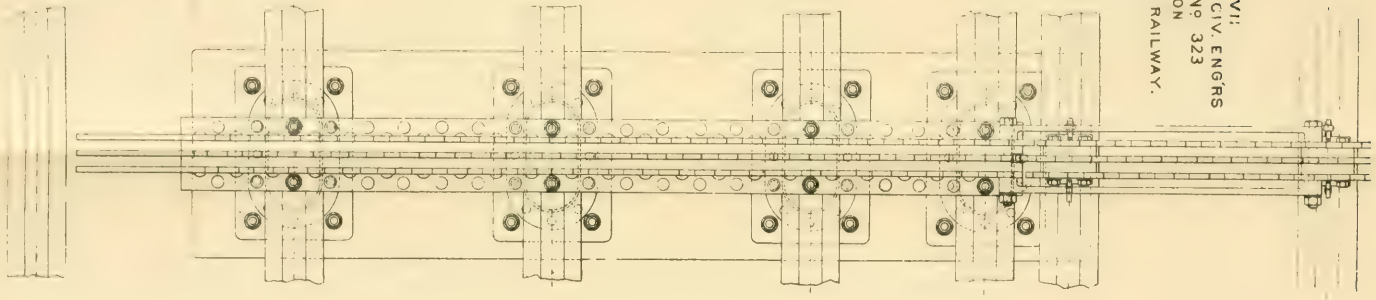


PLATE VI:
TRANS. AM. SOC. C.I.V. ENG'RS
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EVANS ON
ABT SYSTEM OF RAILWAY.

often have to go way back from the sudden ascent, and begin at a point from which, by rising along the mountain sides with expensive cuts, retaining walls and costly tunneling through projecting spurs, we finally reach our aim with 1 in 40, or whatever be our limit. Sometimes a convenient side valley will give a chance for lengthening out the line along its sides, or by switching to and fro in zig-zag line along the slopes, we may obtain the length the gradient requires. In other instances we avoid the delays of the switch-back system by crossing and recrossing a valley from side to side, and in continuous long-stretched loops steadily make the ascent. Then again we boldly cut into the mountain itself, and develop the line in spiral tunnels, the paths often crossing themselves in their plans. A most remarkable example of this latter kind is found, for instance, near Giornico, on the Italian slope of the St. Gothard line. There the road winds itself in two spiral courses up, through, and in the precipitous mountain side, and though the point of ingress and the point where the line last leaves the tunnels are several hundred feet above each other, yet their horizontal distance does not much exceed 40 feet. Still, notwithstanding all such exertions, we often in crossing a range cannot obtain a sufficient altitude, and have to resort to a tunnel of unusual length and cost.

The lower the gradient to which the engineer is limited, the longer will be the line, the greater the call for artificial structures, the deeper the cuts, the higher and costlier the embankments, the more numerous the bridges, and the longer and more frequent the tunnels. Any system which admits an expansion of the limit of gradients will greatly reduce the cost of construction of new roads, but a system like that of Mr. Abt, which combines at once an extensive range of gradients with perfect safety and economy of operation, will prove a perfect boon, one of the most valuable gifts to our modern means of transportation. Through Abt's railway system we can adhere more closely to the bottom of the valleys; in fact we can take the ground much in the shape it is in. All our construction expenses in excavation of earth and rock and in masonry per mile of road will be smaller, the road itself will be materially shorter, and it will take much less time to build it and have it in operation. So great are the savings from all these sources, that, even including the extra expense for rack-rail and engines, the cost per mile of the shorter Abt railway will never reach the cost per mile of the longer adhesion line. Abt's system opens new and vast fields to railway

enterprise; and roads which never could have been thought of on account of enormous expense, can now be built and operated at so much lower cost that they will prove safe and paying investments to capital.

The selection of gradients is a matter of study in each special case. It is as much a function of the natural inclines of the ground as it is one of the general importance of the line, its traffic, and also its total length. On a regular adhesion road with a gradient of 1 in 40, an engine of 48 gross tons in weight will haul on an average a train of 180 gross tons, and the velocity will be from 10 to 14 miles per hour. Our traction table shows that an Abt locomotive can manage the same train of 180 gross tons on a gradient of 5 per cent., or 1 in 20, so that we can say at once, a line with 5 per cent. gradients and the rack-rail handles traffic as readily as one of 2½ per cent. gradients without the rack-rail. No extra engines are required, the same locomotives pull the same train over the entire road.

A principal line, therefore, with 5 per cent. gradients, built and operated by Abt's system, is in every respect equivalent to a regular adhesion line of 2½ per cent. gradients, as far as the handling of the traffic is concerned. But in order to rise from one level into another a line of 1 in 20 will only be half as long as one of 1 in 40.

According to the lengths of the lighter and heavier gradients on a given line, it can be demonstrated that even for principal railways a maximum gradient from 6 to 7 per cent., 317 and 370 feet to the mile may be adopted, and, as a rule, the same traffic be economically managed which is now handled by adhesion lines of 2½ per cent., or 132 feet to the mile. The number of cars, their construction, and the number of locomotives would be the same in either case. The train weight on a 7 per cent. gradient to be sure would only be 130 gross tons, but the proportionately higher speed on the lesser gradients, the resulting saving in time, and the ability of increasing the daily trains in number, fully compensate for the smaller trains.

The velocities on the rack-rail portions, though they appear small at first sight, are, in reality, similar to the velocities on pure adhesion lines. On a 7 per cent. gradient, for instance, our normal rack-rail locomotive will haul a 130-ton train with a speed of about 7.2 miles per hour, but this is equivalent to a speed of over 20 miles per hour on a gradient of 2½ per cent.

A very important point in favor of this new system when compared

with the mountain lines of usual gradients, is the materially increased safety in the working of the line. The hold of the pinion is absolute and positive, the adhesion is only utilized to what the average conditions of the track will warrant, and the facilities for braking are abundant and superior. Then again, the facility with which the trains can pass from adhesion sections to rack-rail sections, admits of arranging stations, depot-yards and switches on horizontal ground, and upon the pure adhesion system. Where desirable, rack-rail switches may be used on gradients, and can be operated with perfect safety.

The general operating expenses are lower with Abt's railway system than with the equivalent adhesion system. The rack-rail sections are considerably shorter than would be the adhesion sections which they replace, and there is correspondingly less maintenance and supervision of the line. There is, then, the additional rack-rail. The life of the rack-rail can be assumed from present experience to be 40 years with a very severe traffic passing over it. Calculating its renewal upon such a basis, and adding the cost of maintenance and ordinary care, which latter expenses are also drawn from the long experience with the unfavorable older form of the ladder-rack, we still get a considerable balance in favor of the rack-rail portions. The consumption of fuel, the expense of firing up, and the losses through stoppages at the stations are the same with either system. But the consumption on the open line is in favor of Abt. The work of lifting train and engine from one level to another one, is, of course, identical in either case; with Abt, however, the higher level is reached by a much shorter route, and the frictional losses, which are in direct proportion to the length of the line, are therefore smaller, and require less fuel to be overcome. It may be claimed that the application of a pinion and rack introduces frictional losses which are wholly unknown to the adhesion system. This claim holds good as far as the friction of the teeth is concerned, but when considering and calculating the frictional losses of the adhesion drivers in curves—and curves preponderate in mountain districts and hilly sections —and when further considering the losses which are due to the unequal wear of the tyres, which happen all along the line, it will be found that the pinion is still a more efficient worker than the ordinary adhesion driver.

With principal or main lines we have thus far spoken only of the application of 7 per cent. gradients; yet circumstances may even here

carry us far higher. If the chief ascent, for instance, occurs at a relatively short section of the line only, we may follow similar examples in the working of adhesion lines, and employ an auxiliary engine. With one engine pulling, and the other one pushing the train, we can carry the same load of 180 gross tons which is hauled on a $2\frac{1}{2}$ per cent. gradient, up an incline of 1 in 10, or 528 feet to the mile. This is certainly doing remarkably well.

In fact two engines are capable of hauling a train of 124 gross tons, or six loaded cars of about 46 000 pounds each, on a gradient of 15 per cent., or 792 feet to the mile = 1 in $6\frac{2}{3}$.

Existing railways with gradients of 150 feet per mile and upwards, will be highly benefited by laying a rack-rail on such gradients. This is an extremely simple matter which requires but very little time. The existing engines can either be rebuilt, or they can be replaced by new ones. The adoption of the rack-rail on such old lines resolves itself into an increased regularity and safety in the operation of the roads, an essentially increased capacity for hauling freight, and sensibly reduced working expenses. It may be stated in a general way that one locomotive of Abt can be made to do the service of two or three adhesion engines. In view of the heavy gradients on many lines of the American continent, it is well to call attention to these things.

With a smaller traffic we make use of smaller engines, and with these latter we can introduce a relatively larger traction, *i. e.*, be able to haul relatively heavier trains. Our traction table relates to what we termed normal types, weighing in complete working order 50 gross tons. In reference to small-sized locomotives, we may mention that an Abt engine is now in course of construction which is to work on a network of narrow-gauge tracks in slate quarries in Bavaria, Germany, and which in complete working order will only weigh $4\frac{1}{2}$ gross tons. We shall speak of that line further on.

Local roads, and especially private lines to factories, quarries, mines, etc., are almost prohibited in hilly or mountainous districts by reason of their first great cost. Economy can only be maintained in a substantial manner by following closely the undulations of the ground, and this is precisely what Abt's system, with its enormous range of gradients, permits us to do.

A very advantageous and extensive application of the rack-rail will be found in connecting factories, mills, etc., with a neighboring

railway station, when either above or below its level. Railroad cars can then be run to and fro in the easiest and cheapest manner on a short rack-railway—a simple truck takes the place of a locomotive, and it is operated by either hand or steam power.

The Harz Railway.* (Plate VIII.)

The "Harz" is the collective term for a massive group of elevations belonging to that last chain of mountains beyond which extend the comparatively low lands and plains of northern Germany. Its medium elevation is about 2 000 feet, and politically it is situated in the Commonwealths of Brunswick and Prussia. Valleys, in the usual sense of the term, there are none to speak of; they partake more of the nature of romantic ravines or gorges with very precipitous slopes. The district contains many valuable mines and quarries, and it abounds in rich woodlands and abundant water power for industrial purposes. Of late years its isolation, by reason of poor means of transportation, made itself more and more felt, and its industries were seriously threatened by not being able to keep their own in the general competition of trade. Several railroad projects were worked out, one strongly supported for strategical reasons, a potent commendation in modern Germany; but they all had to be abandoned on account of the excessive cost of construction. The extent of the Harz district is not large, its greatest dimension being about 20 miles. A through railway, therefore, with its mountainous character and greater cost of operation, can never compete in through traffic with the cheaper, though longer, encircling lines, and it can only count upon the home traffic of the Harz itself. This latter is not large enough, nor will it ever be, to warrant the expense of an ordinary adhesion railway of 1 in 40, or even 1 in 35. Most careful statistical researches of the annual traffic were made, but things looked hopeless enough until Abt's system offered a satisfactory financial solution. The difficulties in locating an adhesion line were greatly aggravated by the fact that the line had to touch the greatest possible number of the scattered industrial settlements, irrespective of the most favorable route from an engineering point of view. All these difficulties disappeared with the freedom Abt's system admits as to gradients.

* In the description of this first actual application of the Abt system, we follow mainly a very able paper (see Glaser's Annalen für Gewerbe und Bauwesen, 1883) by A. Schneider, C. E., Herzoglich Braunschweigischer Bahndirector, who has made all the preliminary studies, and under whose immediate care the road is being built.

The line which is just now in the course of construction, and one-half of which will be opened to traffic this autumn, follows for about 1¾ miles an existing adhesion line from Blankenburg to the blast furnaces. Thence the new road rises rapidly for about 4½ miles to the mines of Hüttenrode. The difference in level is about 810 feet, and, with the exception of a few short horizontal sections, the gradients are 1 in 22, 1 in 20, and 1 in 16.5. From Hüttenrode the line descends with gradients of 1 in 40 and 1 in 100, with two sections of 1 in 20, a total descent of 315 feet in a distance of 2 miles; and ascending again, with gradients not exceeding 1 in 40, it passes the village of Rübeland, with its furnaces and smelting works; then Elbingerode, with its lime quarries, and, after a total rise of 415 feet in a distance of 4¼ miles, the highest point is reached. In the farther descent to the blast furnaces of Rothehütte and the works near Tanne (the terminus) there are two sections with gradients of 1 in 18; otherwise the gradients are 1 in 40, 1 in 60, and 1 in 120, and less. The total length of the railway is not quite 17 miles, and 3.55 miles of this length are provided with the rack-rail. The maximum gradient for the adhesion sections is 1 in 40, and all steeper gradients are provided with the rack-rail. The maximum gradient of the latter is 1 in 16.5, or 320 feet to the mile. The line contains but very few straight sections, and forms in reality a succession of curves. This, in conjunction with the range of gradients, demonstrates forcibly the adaptability of this railway system to the exigencies of the ground. Along the open line the curves have a minimum radius of 655 feet, and in the village of Rübeland there is one of 590 feet. The rack-rail is laid with three elementary bars. Gauge of track, 4 feet 8½ inches. The total estimated cost of this line of 17 miles is $875 000, or about $51 500 per mile.

Extensive field studies and estimates were made in order to establish the difference in cost of construction between a line with pure adhesion gradients and one combined with the rack-rail. These estimates were made for the first 6.7 miles, 3.22 miles of which are laid in 8 different divisions with the rack-rail. These investigations include the minutest details, were carried through with the utmost care, and give the following results, showing the great economical value of the Abt system:

	Abt System.	Adhesion System.
Length	6.7 miles.	9.32 miles.
Real estate required	1 280 000 sq. ft.	1 780 000 sq. ft.

PLATE VIII.
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THE ABT SYSTEM OF RAILWAY

HARZ-RAILWAY

PROFILE

SCALES

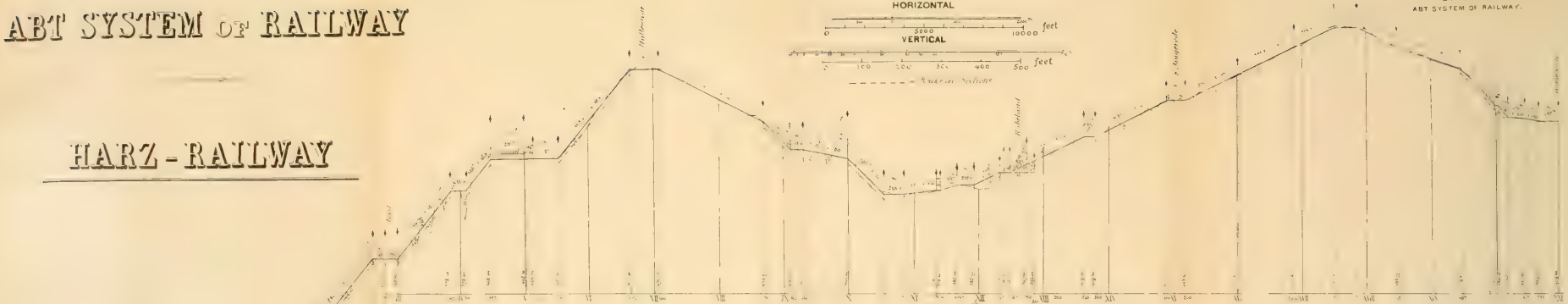
HORIZONTAL

0 5000 10000 feet

VERTICAL

0 300 500 400 600 feet

Broken Section



PLAN

SCALE

0 2000 4000 6000 8000 10000 feet

	Abt System.	Adhesion System.
Excavation (earth and rock),	330 000 cub. yds.	1 120 000 cub. yds.
Masonry (retaining walls, etc.)	5 640 "	33 900 "

The amounts are especially striking in the items of excavation and masonry, the former being about 3½ times and the latter about 6 times larger for the pure adhesion line than for the Abt system. Real estate runs in proportion to the length of the line.

Estimating at the rates there existing, and considering only those construction accounts which differ with the two systems, we find:

		Abt System.	Adhesion.
I.—Acquisition of real estate		$11 875	$16 500
II.	Earthwork, rock, etc	113 825	308 668
	Retaining walls	16 125	97 125
III.—Crossings		20 424	38 845
VI.—Bridges and culverts		2 704	5 414
X.—Superstructure, rails, ties, etc		97 504	97 500
XI.—Signals and telegraphs		405	562
		$262 862	$564 611

All other construction expenses, as for instance, for depot buildings, etc., are alike for both systems. The total saving, therefore, resulting from the application of the Abt system amounts to $301 749, or as the adhesion line would have had a length of $9\frac{30}{100}$ miles, we can say that in this case a saving of about $32 500 per mile of road has been obtained!

According to the stipulations of the contract, the locomotive must haul trains of 120 gross tons, exclusive of engine weight, over the entire road, and maintain a speed of 7½ miles on the 1 in 16.5 grades. The gross traction of the locomotive on this grade is 26 000 pounds, of which one-half is due to the adhesion drivers, and the other half is furnished by the pinion. A general characteristic description of the locomotives of this railway has already been given on page 162. The following are the principal dimensions:

Cylinder dimensions for adhesion	18 × 23½ in.
" " " pinions	12 × 23½ in.
Diameter of drivers	49 in.
Pitch line diameter of pinions	22½ in.
Pressure in boiler above atmosphere	150 pounds.
Heating surface	1 610 square feet.
Grate surface	19.4 "

Total weight in working order	54	gross tons.
Weight on adhesion drivers	42	"
Weight per driving axle	14	"

In designing these locomotives Mr. Abt had to adhere to the Prussian standards; if he could have followed the general locomotive practice the engines would have weighed from 49 to 50 gross tons.

The weight per driving axle may, perhaps, appear a little high, yet weights of 15 to 16 tons are of common occurrence on English lines, and that too with fast passenger engines. Considering the comparatively small velocities on such mountain lines, a weight of 14 tons, which is little influenced by the balancing parts is entirely justified.

Passenger and freight cars are equipped with a continuous train-brake; they are also provided with pinions for braking purposes.

On the 15th and 16th of last May, an experimental train of the regulation 120 gross tons, exclusive of engine weight, was run over the portion of the line which had been sufficiently finished for this purpose. This part contains a gradient of 1 in 16.5, and the change into this gradient takes place from a short horizontal section; on the gradient the line follows a curve. The locomotive maintained with ease the prescribed velocity on this gradient; then the train was stopped, backed, started again, in short it was manœuvred on this incline of 1 in 16.5 with the same facility as on a line of easy gradients. The train was also run with speeds up to 16 miles per hour on the rack-rail, and the working of the pinion in the rack was as noiseless as the working of the adhesion drivers on the ordinary rails. There were no blows, tremors or vibrations in any part, everything worked smoothly and evenly. The entering rack-rail operated to perfection, and the whole was a complete success.

Since then prominent engineers and engineering societies from various parts of the Continent have visited this novel railway, and they all unite in pronouncing it to be one of the most important and far-reaching additions to our modern railway service.

Höllenthal Railway.

A second railway to which Abt's system will be applied is being built by the government of Baden, South Germany. It commences at Freiburg and will extend for the present to Neustadt. Its length is 21¾ miles, 4 miles of which will be laid with the rack-rail; maximum gradient 290 feet

per mile. Gauge of line 4 feet 8½ inches. This road, generally known in Germany as the "Höllenthal Railway," so called from the "Höllenthal," a picturesque and romantic gorge of the Black Forest through which it runs, has been in contemplation for a great number of years. The difficulties of its location, with the adoption of Abt's system, have ceased to exist.

Both these lines, the Harz Railway and the Höllenthal Railway, will be members of the German Railroad Union, and as such, will be open to all continental rolling stock.

Railways in Bavaria.

In Lehesten, Bavaria, extensive slate quarries are being connected with the Bavarian Government Railways by a line upon Abt's system. The gauge is 4 feet 8½ inches, as the rolling stock of the government railroads is to be admitted. The line has a length of 2¾ miles, of which ¾ mile is to be laid with the rack-rail on gradients of 422$\frac{4}{10}$ feet to the mile. The rack-rail is composed of only two elementary bars. The engine will have four adhesion drivers, two driving pinions and two truck wheels. The weight in complete working order will be 20 to 21 gross tons. The normal train weight consists of 50 gross tons, equivalent to three loaded cars. The line will be finished and in working order by November of this year.

At the same time a number of narrow-gauge tracks are being laid in the slate quarries themselves. Gauge, 27½ inches. Maximum gradients for rack-rail, 721 feet to the mile. It is on this network of tracks that the little locomotive of but 4½ gross tons will have to work.

Since writing the foregoing, a translation has been sent to the writer from a German paper, in which Mr. Schneider, Herzoglich Braunschweigischer Bahndirector (under whose official inspection the Harz Railway is being constructed), says:

"All throughout the Harz the name of 'Abt' is mentioned only with a deep feeling of veneration and gratitude, and for time to come many inhabitants of other mountain districts will do likewise."

Notes.

First.—Richard Trevithick, the engineer referred to in the first part of this paper, having made for himself a fine reputation in the matter of steam-pumps, dropped railways, and on being invited to Peru, went there. He succeeded in pumping the water from some rich mines that had been for a long time flooded. The delight of the owners and the enthusiasm of the Viceroy were so great, that a statue of solid silver was

ordered to be erected to him in the Plaza of Lima. Not having seen anything of the kind there, the writer asked what had become of the statue, and was amazed to find that not one of the "Hidalgos" of that once famous city had ever heard of the statue or of Trevithick either. It must be recollected in connection with this, that Trevithick was in Peru in the second decade of this century, not such a long time ago.

Second.—The Panama Railway has been referred to as the line on which George Escol Sellers' center-rail and horizontal driving-wheels were to be first used.

John L. Stephens, the projector and promotor of that line, counted on having to pass over a mountain range and encounter very severe gradients. No line had yet been run nor had accurate levels been taken. At the close of 1848 the writer loaned him one of his assistants, James Baldwin, to run a line and take levels. It was Baldwin who discovered the present location, and the summit of the Panama Railway is known as Baldwin's summit to this day. On this very route Count Ferdinand De Lesseps is now vainly attempting to build a sea-level canal.

Sellers' engines were built for and were the first used on that railway, but they ran only as ordinary adhesion engines. It may be interesting to know that Sellers prosecuted the railway company for not building heavy gradients and using his system, and he actually got a judgment in his favor for $14,000. This information was given to the writer by that Nestor of American engineering, Horatio Allen, Hon. M. Am. Soc. C. E., who was consulting engineer to the Panama Railway Company. It may also be very interesting to the Members of the Society to know that there is still alive and in the possession of great physical and mental ability—now when the world is covered with railways and has taken on itself a new phase of its existence—a man who has walked with George Stephenson over the Liverpool and Manchester Railway during its construction. This man is Horatio Allen. It was he who gave steam and life to the first locomotive ever moved on the American continent; and the writer, your humble servant, had the honor to give steam and life to the first locomotive ever moved south of the equator.

Third.—In April, 1873, Dr. W. Poole, F. R. S., read a paper on the Rigi Railway of Switzerland before the Institution of Civil Engineers of England. In that paper he gives the entire credit of that railway and its peculiar track and rolling stock to Mr. Riggenbach as the inventor. He was not to blame, as he had (the writer has also) seen the words "Système Riggenbach" on the locomotives of that line and elsewhere. Knowing that a most elaborate and detailed report, with numerous graphic pen sketches of every part of the Mount Washington Railway had been made by a Swiss engineer (referred to in the preceding paper), and had been sent to Mr. Riggenbach, the writer saw clearly that Mr. Riggenbach was stealing the thunder that properly belonged to Mr. S. Marsh. The Swiss engineer, Herr Otto Grüninger, kindly consented to give the writer a translation from his German press copy of that report. Another copy the writer had made and bound as a book, to send to the Institution of Civil Engineers in London, where it is on record as evidence of what Mr. Marsh first did with a rack-rail in our time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

324.

(Vol. XV.—March, 1886.)

EARLY SURVEYS AND REPORTS IN REFERENCE TO THE TRANSMISSION OF TRADE ACROSS THE ALLEGHENY MOUNTAINS IN THE STATE OF PENNSYLVANIA, KNOWN AS THE ALLEGHENY PORTAGE.

By MONCURE ROBINSON, Hon. M. Am. Soc. C. E.

NOTE.—Much interest exists as to the earlier engineering thought and work of this country, and the records of this thought and work are not easy of access. Mr. Moncure Robinson, Hon. M. Am. Soc. C. E., who made the original examinations as to the best means for providing accommodation for the transmission of trade over the Allegheny Mountains, between the terminal points of the Pennsylvania State Canals on the eastern and western side of the mountain, has very kindly furnished the Society with copies of his reports. At the request of many Members of the Society, and by permission of Mr. Robinson, they are reprinted in the Transactions. From one of the documents transmitted herewith, the Report of the Canal Commissioners made Decem-

ber 18th, 1829, the following is quoted: "The Portage Railroad across the Allegheny Mountains was included by the Board in the Superintendency of the Western Division, and Moncure Robinson, who had been appointed the engineer in December last, was continued by the present Board for the purpose of completing his examinations and estimates upon this important public work. His report is herewith submitted." * * * * "The best means of effecting this connection between the Eastern and Western waters is a subject of great magnitude, for, independently of other considerations, upon it depends the successful competition of the Pennsylvania Canal with other avenues for the trade of the Western States to the Atlantic. The Board in employing Mr. Robinson upon this work secured the services of an engineer of the first order. They required from him every kind of information necessary to a judicious selection of the best means of effecting the connection. His report justifies the high opinion which was entertained of his talents and professional skill, and gives clear and comprehensive views of the subject."

The responsibility was devolved upon the engineer of recommending the plan of improvement "which might occur to him as calculated to afford the best accommodation to trade," and he was "required to furnish every kind of information necessary to a judicious selection of the best means of effecting the connection."

The result of the examinations was the recommendation by Mr. Robinson of a line of railroad "passing the summit by a tunnel of one mile in length, and overcoming the elevation by five lifts and five levels on each side." In arriving at this conclusion, under the instructions of the Canal Commissioners, topics (especially those growing out of the idea of using the railroad as a public highway) are considered which will appear curious at the present day, but which may be interesting to civil engineers of 1886 who may desire to know something of the difficulties which their predecessors in the profession experienced in propagating scientific or even rational views fifty-seven years ago. In this aspect, and perhaps in others, portions of the reports referred to may be worth a reprint in the Transactions of the Society.

The Legislature of Pennsylvania, in acting on the subject, adopted the plan of a railroad of lifts and levels, and the points of connection with the canals east and west of the Allegheny Mountain (Hollidaysburgh and Johnstown) recommended in the report, but rejected the

plan proposed by the writer for lowering the summit level of the road by a tunnel of a mile in length (the expense and difficulties of which were greatly magnified in the minds of its members), thereby increasing to a corresponding extent the elevation to be overcome, both on the eastern and western slopes of the mountain, and augmenting the difficulty of locating and arranging the planes. The original report is as follows:

H. R. 138.—No. 1.

ALLEGHENY PORTAGE.

Report of Moncure Robinson, Principal Engineer upon the Allegheny Portage.

To the Board of Canal Commissioners of the State of Pennsylvania.

Gentlemen:—By the resolutions of the Board of the 8th of December last, appointing me the engineer of the Allegheny Portage, my attention was directed to three points—"1st, the construction of a railway overcoming the summit by means of stationary engines, or self-acting planes, with intermediate levels, or moderately ascending planes. 2d, The construction of a macadamized turnpike of the best kind between the same points. 3d, The suggestion of any other plan which might occur to me as calculated to afford the best accommodation to trade."

It was further resolved,

"That Mr. Robinson be directed as soon as he shall have made the necessary examinations, to furnish the Board with his views and opinions in detail."

Agreeably to this resolution, I have now the honor to submit to the Board the following report:

Two considerations of less moment in the location of a canal, become objects of indispensable attention in that of a railroad, and in determining its profile, viz., the amount and the direction of the trade to be anticipated on it; a knowledge of each fact, as far as it can be ascertained, is essential to a decision between rival routes, where it becomes often a question how far a more eligible trace may be justified at an enhanced cost. The proportion of trade, or its relative distribution in each direction, has its influence where no such question is to be settled, because on this *ratio* depends the graduation which may be advisable for each portion of the roadway, and which, of course, is to be approached

as nearly as it can be attained without incurring an expense more than commensurate with the object.

In a report of the 4th of December last, on a proposed railway connection between the Susquehanna and Schuylkill, I took occasion to consider this subject in some detail. The conclusion was deduced that between the Susquehanna and the dividing ground—that is to say, in the direction of the greatest transportation—the ascending graduation per mile should be reduced to its minimum, the remaining ascent being overcome by inclined planes and stationary power at points of convenient location. The reasoning which led to this conclusion, applies with greater force to the Allegheny Portage. There can be no doubt that the trade must be immense on a line of railway intended to connect the East and West, and two divisions of canal, each of which passes through a district peculiarly fertile in most valuable but ponderous minerals, and almost as little that whatever may be the present ratio of that trade, that from the West to the East must after a time preponderate.

No views which would be new in confirmation of the opinions here expressed can be offered to the consideration of the Board. Under this impression, they are submitted without comment.

The same views, however, which recommend a cheaper power than that of horses on a railway which is to traverse the Allegheny, and the largest facilities which can be afforded on the graduated portions of it, urge quite as strongly the greatest practical diminution of the amount of absolute power required; or, in other words, that the mountain should be crossed at the least elevation, and by a line of the least length which may be attainable. If, as is believed, the railway will in a few years have nearly as large an amount of trade as it can accommodate, all elevation beyond what is necessarily encountered, any unnecessary increase of distance and obstacles arising from unnecessary curvatures, will form a perpetual tax on this trade, which ought, as far as possible, to be avoided.

Preliminary to the trace of any route, a full examination was made of the summit, as far north and south as there was the least probability that a line would cross. It resulted in ascertaining the fact, that the Sugar Run Summit, to which attention had been first drawn by Mr. Blair, and across which one of the lines of Mr. Roberts had been traced, was the most reduced point within the range of the contemplated connection at which the mountain could be crossed. It next became an

object of inquiry how far a diminished elevation might be attained by tunneling.

Bends of level were traced on each side of the mountain (where the information sought for was not afforded by previous surveys), so as to show by the proximity of these lines when plotted, the points which presented themselves under the most favorable aspect, and examinations were afterwards made on the ground at these points. It was ascertained that south of the nouthern turnpike, a more reduced elevation than that of the Sugar Run Summit could not be effected without incurring the expense of a tunnel of unreasonable length, and more than proportional expense; that north of the Blair's Gap Turnpike, two points presented themselves affording more than ordinary facilities, that at one of them a short distance north of Samuel Lemmon's tavern, a summit level one hundred feet lower than the Sugar Run Summit might be effected by a tunnel of a mile, that at a second point one and a half miles further north, the summit level might be reduced 200 feet by a tunnel of a mile and one-sixth, the greatest height of the comb of earth in the former case being 177 feet, in the latter 240 feet.

Setting aside the point of elevation, it was further apparent on a mere view of the country, that the greatest facilities for the location of a valuable line of railway would not be presented south of the turnpike, and it appeared from the surveys of the preceding season that any practicable line in this quarter could not be materially, if at all, shorter than one passing along the western slope of the Allegheny Mountain, crossing at the point first named, and descending by the Blair's Gap Valley. Under these circumstances it was evident that further examinations might, without hazard, be confined to such lines as would cross the summit either at the Sugar Run Gap, or at one or other of the points above named, as favorable to a reduction of the summit level.

The line first traced was that crossing immediately north of Samuel Lemmon's tavern, this appearing, on a reconnoisance of the ground, to combine more completely than any other the recommendations of reduced elevation, diminished distance and relative facility. A topographical plan and profile of this line, revised at each point, which admitted of any material improvement, is herewith presented. It will be unnecessary to trouble the Board with a detail of the extended and minute examinations which have led to the conclusion that it presents, in an eminent degree, the most eligible route for a railroad on the plan contem-

plated by lifts and levels. It may be proper, however, to remark, that the line which crossed at a more reduced summit was with much reluctance abandoned, and only after it was discovered that it would prove much more expensive; on the western side of the summit much more circuitous, and that insuperable difficulties would attend the location of lifts along the Sugar Run Valley.

The line adopted may, for convenient consideration, be classed in two divisions, commencing at a B M on a beech, on a branch of Clearfield north 45° 30′ west of Lemmons, 2 004 feet above our summit level, and going east it curves on a radius of 500 feet to the western entrance of the tunnel at station 5. After piercing the mountain it is located on the steep slopes south of the north branch of the Blair's Gap Run, falling at five points by inclined planes and in the intervening distances, at a graduation varying between $\frac{3}{100}$ and $\frac{32}{100}$ of a foot per 10 feet. The foot of the last inclined plane is located about 6 miles from the eastern entrance of the tunnel, and about $\frac{1}{3}$ of a mile below the Blair's Gap Inn. From this point the hills fall away on each side of the Blair's Gap Valley, and become too much indented with ravines to admit of retaining such a graduation as would have been preferred. It has been found practicable, however, so to distribute the fall of the valley as to give an average descent per mile from the foot of plane No. 5 east to Hollidaysburgh.

A short distance below this village the railroad will connect with the eastern division of Pennsylvania Canal, and the last 1 538 feet of the line located, terminating at station 308, would be parallel with the proposed basin. Excluding this distance, and computing from S B M I to the head of the basin, the length of the eastern division of the railroad will be 10 miles and 278 poles.

The first 588 poles of the western division of the railroad passes over ground of a very gentle declivity, sloping in the first instance towards Storm's Run, a tributary of Clearfield, and afterwards towards the Laurel Swamp branch of the Conemaugh. At the end of this distance the inclined planes west of the summit commence, the line falling successively at Adam's Run, Bear Rock Run and Ben's Creek. A fourth inclined plane is located about $\frac{3}{4}$ of a mile below Litzinger's saw mill, and lowers the line into the valley of Conemaugh.

The graduation between these planes varies between $\frac{11}{100}$ and $\frac{32}{100}$ of a foot for each distance of 10 feet, and may be made somewhat more gentle on a definitive location.

Station 214, at the foot of inclined plane No. 4, is 8 miles and 191 poles from S B M I. From this point to the big bend of the Conemaugh, a distance of 9 miles and 269 poles, the line is located (with the exception of a short distance from the foot of plane No. 4) on the north side of the Conemaugh, and is graduated on a slope generally rising to, but in no case exceeding 13 feet and $\frac{2}{10}$ per mile. At this point plane No. 5 west is located, by which the line is lowered 91½ feet. Between the foot of this plane and Johnstown, 9 miles and 20 poles, the Conemaugh is crossed five times, so as to obtain for the railroad the best and most direct line. The trace in this distance, nevertheless, deviates more from a straight line than on any portion of the route; on this part of the location the graduation is for the greater part of the distance as steep as 26 feet and $\frac{40}{100}$ per mile, but at no point exceeds this descent.

The line of the railroad terminates at No. 671, near Johnstown, the last 1 300 feet being at a convenient distance from an arm of the Conemaugh, which will be converted into a large and commodious basin by raising Livergood's dam and embanking on the opposite flat; deducting this distance, the whole length of the western division of the railroad from S B M I to the head of the proposed basin, is 27 miles and 93 poles, and the whole distance between the two basins 38 miles and 51 poles.

The points adopted as points of termination for the railroad and of connection with the canals east and west of the mountain, are recommended after much consideration. It will be proper to explain to the Board at some length, the reasons which have led to the conclusion that they are on the whole the most advisable.

It will be observed, on examination, that the profile of the western part of the proposed railroad from the foot of inclined plane No. 4, to its point of termination, is well adapted to the use of locomotive engines. A motive power may in consequence be adopted on this part of the railroad peculiarly eligible where expedition becomes an object, and better adapted than any other to the case of a profile within the limits of locomotive graduation, and which for other power would not be advantageous. On the other hand there is little doubt that an ample supply of water for locks of even the small lift of 5 feet cannot be commanded from the Conemaugh above the confluence of the south branch. This circumstance would have appeared decisive against an extension of the canal above Johnstown, had it been clear that an increased quantity

could not have been obtained elsewhere. It was deemed proper, however, to omit no view of the subject which could with propriety be taken under the instructions given, and which might be desired by the Board. Examinations were accordingly commenced with the view of ascertaining what further supply could be commanded at the confluence of the Munster and Ebensburg branches.

It is ascertained that a feeder, which would not be expensive, for 5½ miles, may be taken from the Bedford and Cedar swamp branches of the south branch, immediately above their junction and brought without any material difficulty to a depressed point in the dividing ground between the Conemaugh and south branch. From this point (a little south of William Brookbank's on the Johnstown road) it would be necessary to conduct the water by a vault of 5 feet diameter a distance of 1 000 yards, after which it would be discharged into one of the sources of Ephraim's run, and thence find its way along the bed of that stream to the Conemagh, 2 miles above the point in question. A small portion of the waters of the Clearfield might also be commanded without much difficulty, and at several points on the Conemaugh, reservoirs of large capacity could, if requisite, be constructed.

Under these circumstances, it may be deemed perfectly practicable to extend the navigation west of the dividing ground, by locks of any convenient size, 17 miles above Johnstown, and it is not to be denied that some very material advantages would flow from such a reduction in the length of the portage. It is believed, however, that these will be overbalanced.

First.—By the increased expense of a canal in this distance; locks constructed on any permanent plan would cost as much, or nearly as much, as the railway. To this is to be added the cost of a canal embracing much difficult ground and of a feeder involving a subterraneous passage of the length above stated.

Secondly.—By the delay attendant on passing through as many locks as would be necessary to overcome the fall (400 feet) in this distance.

Thirdly.—By the increased annual expenses on the canal when completed. This item for repairs and renewals alone might, perhaps, be equal on the canal and railroad, but a large number of officers would be required on the former work, who might be dispensed with on the latter. Their salaries at the lowest rates at which competent officers could be procured, would form a percentage on the tolls which might

be anticipated worthy of much consideration in an estimate of the relative value of either improvement.

Fourthly.—By the interruptions to a continued communication between the east and west, which might be anticipated on this part of the line.

These would arise from two causes: 1st. From the peculiar liability of a canal presenting so large an amount of lockage in a short distance to derangement; and 2dly, the obstruction of navigation by ice at a later period in the spring and at an earlier period in the fall of the year than on other parts of the route. This last effect would result not only from the greater elevation of this part of the line, but the peculiarly confined character of the Conemaugh valley in this distance.

The subject, however, is submitted with due deference to the better judgment of the Board. Should it be their opinion that the advantages of pushing canal navigation west of the Allegheny to the highest practicable point preponderate over the disadvantages here exposed, it may be expedient to adopt the location of the railroad only as far as plane No. 4 west; and to cause a particular estimate of the expense of a feeder from the south branch, and a canal the remaining distance to Johnstown to be made before coming to a definite decision.

The eastern termination of the railroad appears to be a matter of much less doubt. A basin immediately above Frankstown would be more easily supplied with water from the Juniata below the junction of the Beaver dam and southwest branches, but would possess no other recommendation. The situation is represented as peculiarly unhealthy, and the only building ground would be either a very low flat or steep hill-side. Should the point at which canal navigation will terminate east of the mountain be, as it is believed it will be in time, a large and flourishing depot, some better accommodation would seem to be required for the trade and the inhabitants, whose pursuits may lead them to settle near it. The same supply of water which can be commanded at Frankstown, may be commanded by the feeder lines laid down in the plan herewith submitted, at a point peculiarly picturesque, said to be uncommonly healthy, and presenting every advantage for the head of canal navigation; and one to which it may be extended with a very slight increase of lockage.

In the description of the line of the railroad, a mere outline of the trace adopted has been given. The consideration of the points at which

stationary power must be adopted, and those at which inclined planes may be admissible, has been reserved; because a previous discussion of some points, and an exposition of certain principles on which this will depend will be necessary.

Wherever on a line of railway a trade can be regulated, self-acting planes are available. If the line of railway descend, and the trade preponderate in the direction of the descent, stationary power at inclined planes becomes altogether unnecessary. If a summit is to be crossed, stationary power must be made use of to an extent sufficient to overcome the preponderance of the heavier trade, and the friction of the machinery employed in raising it; but this heavier trade after ascending, may be made use of to lift up a lighter returning trade.

Where, of course, a trade on a line of railway is nearly equal, power to a certain extent becomes indispensable at each plane, in either direction from the summit. Its amount may, however, in all cases be materially diminished by such a regulation of the trade as will bring trains of wagons to the foot and head of planes at the same time.

There can be but little doubt that the trade on the contemplated line of railway must be in a short time a much heavier one from west to east, than in the opposite direction. The bituminous coal of the Allegheny, which even now is taken occasionally over a very bad road to the villages on the Juniata, will probably of itself be equal in weight to the whole amount of iron and merchandise returning.

Under these circumstances, powerful steam engines (should steam be the power adopted) will be required at planes Nos. 1, 2, 3, 4 and 5 west of the summit, but if the trade be a regulated trade, the gravity of descending trains will be nearly or quite sufficient east of it.

Such a result may be attained in two ways. Either the privilege of transportation may be leased for a term of years to individuals or a company, who should transport at fixed rates, or it may be effected by agents of the commonwealth at specified rates for toll and transportation.

I am aware of the preference which will always be given in Pennsylvania to public highways, on which the fullest competition may have play, and that very plausible objections may be made to either of the plans suggested. It is believed, however, that one or other disposition of the portage, at any rate for some years to come, is recommended by considerations of paramount weight. 1st. By the diminished amount

of power required for any given amount of business, should the proposed railroad be a public highway, 5 steam engines averaging 70 horse power each, at an annual expense in fuel, wear and tear, and attendance, of $15 000, or other stationary power equivalent to them, will be required in addition to those stated to be necessary west of the summit; and at least ⅓ more horse power will be necessary on those portions of the railroad on which it is employed. To understand this remark, it will be necessary to advert to the circumstance that a horse can for a short time sustain a stress of which he would not be capable for a series of hours. Should transportation on the contemplated railway be effected by lessees or agents of the commonwealth, horses operating in relays would be used on the short levels of the road, whilst locomotive engines would be employed on the longer. On these short levels a horse would be capable of a very great effort in ascending, because this effort would be of short duration, and he would be relieved in returning. It would be otherwise, however, if it was a question of traveling the whole length of the railroad, ascending and descending in a trip an elevation of 2 270 feet. His load, it will be obvious in this case, could not very greatly exceed what would be his fair load from his starting point to the summit level.

Secondly.—By the opportunity which it affords of dispensing with crossing places and turnouts between the inclined planes. These on a line of railroad in a mountainous country, add materially to its expense, in consequence of the increased width of roadway they require, occasionally at different points and on steep slopes, but could not be dispensed with without so regulating the traffic as that trains traveling in either direction would progress with equal speed.

Thirdly.—By the superior economy, as well as expedition of locomotive power. This, of course, must be given up between plane No. 5 west and Johnstown, should the railroad be a public highway, as locomotive engines and horses could not be advantageously made use of on the same parts of the road. A further inconvenience would result from this circumstance, which may require some explanation.

The trade on the contemplated improvement must necessarily be irregular. During three or four months of the year at least, the canals may be expected to be bound up by ice, and at midsummer there will probably be but little trade to or from any point beyond Pittsburg, in consequence of the difficulties attending the navigation of the Ohio at

that season. It can scarcely be expected that any competition on the proposed railroad, if a public highway, would insure adequate means of transportation for what might accumulate at each extremity of the road during those periods of the year when all the avenues of trade would be open. This end, however, will be attained if transportation is effected by the commonwealth, or by individuals to whom the privilege shall have been leased. In the latter case conditions may be affixed and requisitions made, which will be easily complied with, both because a given amount of power will be productive of a larger result under this than under any other arrangement; and because a surplus number of locomotive engines may be retained on those parts of the road on which they can be used, without incurring any further expense when they may be unemployed than the amount of interest on their cost.

It remains to be added, that should transportation on the proposed railroad be effected by agents, or responsible lessees of the commonwealth, the objections which have been made to the Pennsylvania line of communication, as presenting in its portage danger of delay and uncertainty, will be in a great measure avoided; and it may not be digressing too far or hazarding too much, to express the confident belief that under such a system transportation may be effected, not only more expeditiously, but as cheaply and with more certainty by the portage than it could have been by a water communication across the mountain, had this last been attainable.

The annexed paper, marked A, presents a description, and an estimate in detail of each division of the railroad, divided into natural sections. It will be proper in this place to discuss the style of execution of the proposed improvement.

The width of the roadway formation will be 21 feet. This will admit of two tracks of 5 feet width each, an intervening space of 3 feet, a foot-path and drain on sloping ground; or of two tracks and two foot-paths on embankments. In the few cases of deep cuttings which occur, an extra width proportioned to the length and depth of the cut will be required.

Embankments, when but small vents are requisite to pass streams or spring torrents, are generally preferred to bridges, and in the formation of inclined planes where the profile of the surface is much depressed below that of the plane, the requisite graduation is attained by embankments or walling, and in no case by trusses. Undoubtedly a considerable sav-

ing in first cost would accrue from the substitution of wooden bridges, and frames of timber for the heavy walls and embankments which are contemplated. When it is recollected, however, that the great weight of a train of railroad wagons would expose such structures to a stress under which they would occasionally yield, and that the destruction of one of them from this cause, or by a mischievous incendiary, might produce an interruption to trade during weeks or months, the propriety of dispensing with them, except under particular circumstances, will be at once perceived.

In the superstructure of the railroad more economy may be exercised than in grading or roadway formation. The cheapness of timber, and the facility with which they may be renewed, will recommend, in the first instance at any rate, wooden rails plated with iron bars in preference to rails of malleable or cast-iron. A superstructure of this description is recommended by the further consideration that a less expensive description of iron may be made use of for plating wooden rails than would be required for rails entirely of metal.

Of course, on those parts of the railroad on which steam power is contemplated, some extra strength will be requisite. The wooden rails must be stouter, and their points of support more frequent than on other portions of the work. It will also be proper to give to the plate rails a somewhat different section and some increase of weight. On the other hand some diminution of expense will be occasioned by dispensing with a horse-path on this portion of the roadway.

The following construction is recommended for the superstructure of the railroad (except at inclined planes) between Hollidaysburgh and the foot of inclined plane No. 4, west, or on that part of the railroad on which horse power is contemplated.

Blocks of stone 2 feet 6 inches long, 2 feet deep and 15 inches wide, to be embedded every 8½ feet apart on a layer of broken stone of the depth of 6 inches. Each of these blocks to be drilled at 2 points to the depth of 8 inches, and its surface to be leveled with that of the roadway. On them blocks of white oak or locust notched for the reception of rails to be attached with locust trunnels. The ends and points of support of the wooden rails to be secured in these blocks by a key so as to admit of removal, raising or shifting to the one side or other of the groove in the simplest manner. The advantages of this construction are believed to be material; not only the rail is elevated and less exposed to the operation

of causes which produce decay, but the inconvenience arising from a slight settling of the foundation may be at once remedied by a removal of the key and raising the rail, or if the parallelism of the railway should be disturbed, by shifting the rail laterally.

The least weight to which it has so far been found expedient to reduce locomotive engines, including the fuel and water with which they should be furnished at each point of supply, is 6½ tons, or about 2½ tons more than the weight of the cars and their loads which are contemplated on the railroad. To meet this increased stress, the following construction is proposed on those parts of the railroad on which machinery will be employed; or between plane No. 4, west, and Johnstown.

Two feet cubes of stone to be embedded as before on broken stone, 7 feet apart from center to center in the direction of the railroad; and on these cubes, sills of white oak or locust extending across the track to be bolted. The wooden rails to be keyed into the sills as in the former instance into the blocks. The only difference in the principle of the two constructions, it will be observed, consists in extending the sills across the railroad track. This construction, which would not be advisable on other parts of the railroad on account of its interference with the horse-path, is recommended on that portion on which locomotive engines will operate, on account of its superior solidity.

Rails 6½ inches by 10 inches, and plate rails half an inch by two inches, are proposed for those parts of the railroad on which horse power will be employed. For the remainder of the road and the superstructure of inclined planes, rails 8 by 12 inches, and plate rails having a cross-section equal to $1\frac{4}{10}$ inches, will be required.

The whole cost of the contemplated improvement, it will be observed, is estimated at $936 004.87. The prices allowed are deemed liberal, and believed to be sufficient to execute each description of work involved in the railroad in the most substantial manner. It would of course be unwise in the extreme to execute otherwise a line of communication on which not only the value of two great divisions of canal, but of many other improvements in the State must essentially depend.

Next in order to "the construction of a railway between the waters of Juniata and Conemaugh, overcoming the summit by means of stationary engines, and self-acting planes," attention was directed to the construction of a macadamized turnpike of the best kind between the same points.

No examinations have been made especially in reference to this object during the past season, nor do any in addition to the surveys made by authority of the State in the summer of 1828, appear to be required. It results from these that a tolerably direct route within the limit of one degree of graduation cannot be had between the points of contemplated connection, and that whilst a railroad of the most advantageous description between Johnstown and Hollidaysburgh will not much exceed thirty-eight miles, a macadamized turnpike of the graduation above stated, between Johnstown and the head of the basin contemplated at Frankstown (only two miles lower down), cannot fall much, if at all, short of fifty.

It may sometimes be a question whether an inferior improvement of diminished length should not be preferred to one of superior order, but by which the distance between two points would be materially increased. When, however, the reverse is the case, and the trade to be accommodated is considerable, such a question we should think could very rarely arise. It will be proper, however, to institute a more precise comparison between the two kinds of improvement submitted.

Such a comparison involves three points: the first cost, the annual expense, and the cost of transportation on each species of improvement.

The estimates of Mr. Roberts (see Canal Documents, 1828, p. 216) give $506 145.34 as the cost of a macadamized road 36 feet wide, on the shortest of the routes surveyed by him at a graduation of one degree. To this estimate remains to be added an allowance for contingencies and superintendence, so that the whole cost of such a road may be stated without danger of excess at at least $550 000 between Johnstown and Frankstown, or at $528 000 between Johnstown and Hollidaysburgh. This would leave a difference in cost between the macadamized road and railroad of $408 000.

The difference in their annual expense is a matter of more difficult determination. Whilst the necessary expenses of a railroad are under any circumstances considerable, they increase in a very slight ratio with the increase of its trade. Those of a turnpike, on the other hand, increase in nearly the same ratio. It certainly will not be rating too highly the business which may be anticipated on the Allegheny Portage to suppose that the annual repairs of a turnpike which would accommodate it, would at least equal those of repairs and renewal (setting aside the expenses of fixed and locomotive power) on a railroad. If the

supposition be correct, it will remain only to compare the cost of transportation on the railroad and turnpike with the first cost of the respective improvements.

If the following calculations are correct (and no doubt is entertained that they are not too favorable to the railroad), they show that with a trade limited certainly in comparison with what may be anticipated, it would form, notwithstanding its increased cost, much the most advisable connection.

Estimating the trade between Johnstown and Hollidaysburgh at 30 000 tons going eastwardly—the trade going westwardly to be taken in return loads—and that transportation is effected during 250 days in the year, the average transportation eastwardly would be 120 tons per diem.

The average load on a horse, on a macadamized turnpike, would be about twelve cwt., or $\frac{3}{5}$ths of a ton. The time occupied in a trip 7 days, viz.: 5 going and returning, and one at each point of termination for rest and loading.

Assuming these data $120 \times \frac{5}{3} \times 7 = 1\,400$ horses would be required on a turnpike between Johnston and Hollidaysburgh; and estimating 5 horses a team, 280 drivers.

Five locomotive engines, forty horses and thirty drivers and tenders, would more completely accommodate the same trade on a railroad. Presuming the wear and tear and interest on turnpike and railroad wagons to be equal, and estimating the daily expense of a locomotive engine as equal to the interest on the cost, the wear and tear and expense of five horses, the saving in motive power would be equal to the wear and tear, expense and interest on the cost of 1 335 horses and 250 drivers.

The former item of expense could scarcely be less than 30 cents per day per horse during the whole year, and an average of 60 cents per day for faithful and trusty drivers would probably not exceed what their services would command.

Then $1\,335 \times 30 + 250 \times 60 = 500$ dollars and 50 cents per day, or $200-932.50 per annum, would be the difference in the expense of transportation on a turnpike and macadamized road.

Deducting from this amount the expense of fuel and attendance for five stationary engines, estimated at $2 200 each ($11 000) and of attendance at five self-acting planes, at $5 000, we have still left in favor of the railroad a saving in the annual cost of transportation of $184 932.52, or a sum equal to nine times the interest on the difference in the first cost

of the two improvements. The commonwealth may of course indemnify herself largely for any additional outlay she may make, and yet leave her citizens largely the gainers.

It will be seen at once, that with a trade greater than that on which the foregoing calculation is predicated, the balance in favor of the railroad would not only be greater, but increased in a still greater ratio because the expenses on the railroad for fixed power and attendance at planes would be very slightly increased, whilst those for repairs on the turnpike would increase in a ratio nearly correspondent with its increase of business.

It may not be out of place here to correct some misapprehensions which appear to exist in relation to the value of macadamized roads as a means of transportation for merchandize and produce.

Travelers who have visited Great Britain have been struck with the smoothness and easy graduation of the roads which traverse every portion of that highly improved country, and have returned to America enamored of an improvement which owes its success rather to the circumstances under which it was introduced, than to the character of the improvement itself. It was long after canals had been constructed in every quarter of England, and the benefits of improved communication been extended into its more broken and mountainous districts by railroads, that macadamized roads were introduced. They were wanted as a means of accommodation for the greatly increased traveling which had been occasioned by an immensely extended internal and external commerce, and as such, have answered in an admirable degree the end for which they were desired. It would have been otherwise, however, had they been made use of to effect the heavy transportation of the interior of the country, or to convey to the seaboard the products of its mines and manufactories, destined for foreign ports.

The small fragments of stone of which it is essential that the covering of macadamized roads should consist, triturate very rapidly even under the light weights conveyed on them in England. Although it is extremely rare that any heavier vehicle than a stage coach is seen on an English turnpike, yet the annual expense of repairs on those which are much traveled, is estimated at £100 sterling per mile.

Were the carriages used on them heavier, their capping would crush in pieces much more rapidly. The subject has been ingeniously illustrated by a comparison drawn from the effect of a large and small

hammer. A pebble, it has been correctly observed, which would resist a severe blow from a hammer of two pounds, would be crushed into powder by one of moderate intensity from a hammer of four pounds.

The repairs of a macadamized road, on which heavy wagons were used, would not only be extremely expensive from the excessive destruction of materials, but also from the constant labor which would be required to fill up the ruts in the road which the wheels of such wagons would be continually wearing. No plan which has yet been fallen on will avoid this on a good road.

Large wheels, on a bad road, will avoid the deepening of ruts beyond a certain point, but if the materials of which the road is composed are small, they must yield under pressure, and the evil is only aggravated with large wheels, because such wheels cannot press uniformly on a rounded surface, which a macadamized road should always have; and would cut more deeply at their corners than narrower wheels, which would have a more equal bearing.

It is proper to mention before leaving this subject, that good materials for the capping of a macadamized road would be had with difficulty in the neighborhood of either of the lines surveyed. The sandstone of the Allegheny is sometimes hard, but pulverizes easily under pressure, and bonds together but indifferently.

The third point to which attention was directed by the resolution of the Board, was "the suggestion of any other plan which might be deemed best calculated to accommodate trade."

Under this head it may be observed that next to a good water communication, no plan is in general deemed more eligible for the accommodation of a large trade than a railroad, overcoming elevations by means of stationary power and inclined planes.

The profile of the line of railroad surveyed across the Allegheny Mountain, may certainly be considered in a high degree favorable. More than two-thirds of the distance between Johnstown and the summit level, as has been before observed, is well adapted to the use of locomotive power. On the remaining third, the ascending graduation is but small, and may be still further reduced by a slight increase in the rise of each lift, without any material accession of expense. East of the summit the profile of the railroad is not quite as favorable as west of it, but on the whole may not be considered ineligible for a trade preponderating in this direction.

The plan of the road is not more unfavorable than its profile. At two or three points it may be advisable to turn on a smaller radius than five hundred feet, but short and sudden curvatures will be but very seldom required.

Nature, then, has not shown herself churlish in the distribution of facilities for the contemplated connection, and whilst she has maintained her barrier of partition between the waters of the Chesapeake and Mississippi, at the head of the Juniata, as at most other points, unbroken, she has presented perhaps as many facilities in this quarter for overcoming it by an improvement of the best description, as could reasonably have been expected.

On the other hand, she appears with equal energy and a boldness scarcely to be mistaken, to have denounced any plan of effecting this object by graduation. The wide and deep ravines with which both the eastern and western slopes of the Allegheny are torn, and which have been found so advantageous for the location of inclined planes, would have presented obstacles to a graded road of any kind, which could only have been overcome at an enormous expense, or avoided by a very great increase in the length of a line.

Under these circumstances, it is not hazarding too much to say, that no plan of improvement can be offered to the consideration of the Board more eligible than that submitted under the first head of this report. It may, however, be a question whether a cheaper and simpler power cannot be substituted at inclined planes for that of steam, which has been contemplated in the annexed estimate.

Water power is available with but little fall, wherever a sufficiently large body can be commanded. It is not the less so where the reverse is the case, that is to say, where the volume of water, though small, can be commanded from a very great elevation.

In but one case on the line of the railroad can a sufficiently large body of water be commanded at an inclined plane, to act in the ordinary manner on a water wheel. At the Big Bend of the Conemaugh, the whole stream may be had at a point half way down the plane, with the advantage of a head and fall of fifty feet, if necessary; affording, of course, for the inclined plane at this point, a greater amount of power than can in any contingency be wanted.

At the other inclined planes west of the summit, considerable streams of water could not be commanded; but at three of them, small and con-

stant streams may be brought, without encountering any very serious expense, to the head or midway of the planes. Two plans are deemed worthy of suggestion to the Board by which the power they would afford may be made use of.

The first plan purposes a piston, cylinder and crank, in all respects similar to those of the steam engine. The water to be conveyed by pipes with the advantage of the greatest head which can conveniently be had, and to be admitted alternately above and below the piston (as steam from the boiler of a steam engine), by means of a four-way-cock or valves; the latter to be opened and shut as in the steam engine, by means of a rod attached to the working beam. The principle on which this machinery (usually denominated the water-pressure engine) is dependent for its power, it will be observed is the same as that of the hydrostatic press of Bramah, from which such important results have of late years been derived.

The second plan purposes the use of water in cisterns as a counterpoise to ascending weights, reducing the operation of ascending west as well as east of the summit, to that of self-acting planes. The annexed drawing and specifications marked B, exhibit its details. It is proper to observe to the Board, that this plan of overcoming elevations is claimed as an invention of the undersigned, and under a favorable impression of its applicability under many circumstances, was secured by patent before he had the honor of being engaged in their service. He will, of course, be excused from any comparison of its advantages or disadvantages with those of the other plans submitted.

It may be desirable before closing this report, to present to the view of the Board, some considerations which have governed in relation to the location of inclined planes on the proposed railroad, which it has not been found convenient before to introduce.

Two points have been deemed to be of essential importance in the location of inclined planes. First, that they should be straight, and that the line of the railroad should, for a short distance from the head and foot of each plane, be in the same direction with the plane; and secondly, that the plane itself should either have a uniform inclination with the horizon, or an inclination gradually diminishing between the head and foot of the plane. On a plane to any extent curved in its plan, a greater loss of power and wear and tear of cable and rope will always be sustained than on a straight plane, and without a stage of some extent at

the head of the plane in the same direction with the plane, a very extensive business cannot be accommodated; and should the plane be so far curved as to prevent the manager at the head of the plane from embracing its whole extent under his view, the operation of raising and lowering trains must be conducted by signals, and often disadvantageously.

On planes on which power is to be applied, an inclination slightly diminishing towards the foot of the plane is most desirable; on self-acting planes (where the fullest useful effect of the gravity of descending bodies is desired) the most advisable profile is an approximation to the cycloid. It is in either case a disadvantageous feature, if the plane becomes steeper towards its foot. In the one case, the engine, at the commencement of its labor, is compelled to exert a greater force than is afterwards required. In the other, a descending train of wagons may acquire, in consequence of its increased stress, combined with its accelerating force, a momentum not to be controlled without difficulty by a brake, and attended with much hazard.

It was farther an object in the case under consideration, to reduce the number of planes as far as practicable. Such a reduction was peculiarly recommended by the saving of time in transportation which it insured, and the superior simplicity of any system of transportation which might be adopted on the completion of the work.

The bold outlines of the country over which the railroad has been located, did not admit of prescribing to it too many conditions. Of course, in complying with those which were deemed essential, secondary considerations have been occasionally waived.

The accompanying volume of profiles marked C, presents a view of the ground which has been selected for the different planes east and west of the summit, and the general inclination recommended at each plane. This of course does not correspond in all cases with that which, looking only to the trade and its direction, would be deemed the most advisable, but deviates as little from such a profile as a due regard to circumstances would permit. The great essentials in the location of planes above explained, it has been found practicable, with one exception, to observe. Plane No. 2 east has a curvature in its course corresponding to a versed sine of 40 feet on a chord of 672 feet. It may be proper to mention also, that at plane No. 5 west, it will be necessary to arrive on and leave the foot of the plane by a turning platform.

It will be observed that the elevation overcome, and angle of inclination adopted at the different lifts, differ very materially. These differences would be compensated on the execution of the work by adopting at each point machinery of suitable weight and power. The proportions allowed in the estimate are deemed a sufficient approximation for the purpose for which they are used, but a nice calculation of strength for each portion of the machinery employed, will of course be requisite, in order to avoid on the one hand the loss of power which would be sustained by the use of cables and other fixtures unnecessarily cumbrous, or the disastrous consequences which might result from the want of a due degree of strength in their proportions.

In addition to the documents above referred to, two volumes of topographical sketches, and two field books, presenting all further information on the subject believed to be material, are herewith presented to the Board.

All which is respectfully submitted,

MONCURE ROBINSON,

Civil Engineer.

November 21st, 1829.

The following portion of the estimate accompanying the above report may not be uninteresting at the present day:

ESTIMATED COST OF CONSTRUCTING THE PORTAGE RAILROAD, ETC.

Average Expense of a Mile of Railroad adapted to Horse Power.

2 560 stone blocks, embedded and drilled, at 75 cents	$1 920 00
Same number of wooden blocks (white oak or locust), 12 inches square, and trunneled to stones, at 40 cents	1 024 00
Same number of wedges or keys, at 5 cents	128 00
21 824 lineal feet of wooden rails, 6½ by 10 inches, allowing 3⅓ per cent. for waste, at 4 cents	872 96
21 200 feet of iron rails, 2 inches by ½ inch, weight 3⅓ pounds per foot, 31.55 tons, at $75 per ton	2 366 25
Four-inch spikes	191 25
Strips of iron or zinc under ends of rails	36 54
Placing, dressing and keying wooden rails of 320 lineal perches of railway, at 60 cents per perch	192 00
Putting down and spiking 320 poles of iron rails, at 70 cents	224 00
Stoning horse-path, 640 cubic yards of small broken stone, at 75 cents per yard	480 00
	$7 435 00

Average Expense of a Mile of Railroad adapted to Locomotive Power.

3 018 stone blocks (cubes of 2 feet), embedded, drilled and plugged for the reception of bolts, at 75 cents	$2 263 50
1 510 sills of locust or white oak, 9 feet long, 12 by 12, sawed for the reception of rails and bolted, at 80 cents	1 208 00
3 018 bolts, 20 inches long, by one inch in diameter, at 30 cents	905 40
3 018 keys, at 5 cents	150 90
21 824 lineal feet of wooden rails, 8 by 12, at 6 cents	1 309 44
21 200 lineal feet of iron rails, cross-section equal to 1.41 inches, weight 44$\frac{3}{10}$ tons, at $75	3,322 50
Wrought-iron spikes, 5½ inches long	225 00
Fixtures at ends of rails	53 26
Placing, dressing and keying wooden rails of 320 perches of railway, at 80 cents per perch	256 00
Putting down and spiking 320 poles of iron rails, at same rate	256 00
	$9 950 00

During the year following the presentation of the above report, a line of different characteristics from that above described was traced and recommended by Lieut.-Col. Long. At the request of the Canal Commissioners a second report was made by the writer which is given below:

To the Board of Canal Commissioners of the State of Pennsylvania:

GENTLEMEN:—It appears that the several engineers appointed under the Act of the 27th of March last, to inspect the several routes and report in relation to the Allegheny Portage, concur in deeming a railroad preferable to a macadamized road, and in awarding their preference to "a route leading upwards along the valleys of Blair's Gap run, crossing the Allegheny Mountain at Blair's Gap summit, and descending to Johnstown, in the valleys of Laurel Run and the Little Conemaugh;" but that two lines complying with the above conditions are at issue before them; the one traced by the undersigned in 1829, and by him recommended to the Canal Commissioners in his report of that year; the other traced by Lieut.-Col. Long in the summer and fall of 1830.

As the opinions of the undersigned in regard to the superior eligibility of the line traced by him in 1829 remain unchanged, after a careful review of all the considerations which appear to him to have an influence in the selection of a route, it now becomes his duty to say so, and to refer the Board of Canal Commissioners to his report of 1829, and the accompanying documents for an exposition of his plans in detail.* In doing so he does not mean to say that on a review of the line many improvements on particular portions of it might not be made. On the contrary, he has, in the report above alluded to, pointed out changes which it would be desirable to effect, and alluded to blemishes which, on a final adjustment of the line, it would be advantageous to avoid. But he is satisfied that the line in all its important features will be found the most advisable, and particularly in those by which it is distinguished from the line traced during the past season by Lieut.-Col. Long.

The undersigned does not mean to enter into an unnecessary and invidious analysis of the two lines. He deems it, however, under the circumstances stated, due to the Canal Commissioners and to himself, as well as to the great interests which rest on the execution of the proposed

* See Register, volume 5th, pages 97 and 116.

improvement in the most judicious manner, to compare them in those respects in which they most widely differ, and to present in as brief a manner as possible some views which appear to him decisive on the subject.

The line traced by Col. Long appears to be the result of "a course of examination and surveys, the leading objects of which were to obviate the necessity of constructing a tunnel at the summit till such time as the exigencies of the trade upon the road should require it; to avoid the adoption of inclined planes having inclination greater than 5 degrees, and, if possible, to limit their inclinations to 3 degrees; to provide for a route upon which not only stationary, but progressive power might operate to advantage, according to circumstances; to effect a definite location of a route upon ground most favorable for the construction of a road in regard to the principles, as well as to the cost of construction," etc.

The results of the course of examinations and surveys of Col. Long are, that the necessity of a tunnel at the summit is obviated by encountering that very summit which the tunnel was recommended to avoid, with the exception of "a deep cut about 1 500 feet long, the greatest depth of which will be 18 feet." In avoiding the adoption of inclined planes having inclinations greater than 5 degrees, and in limiting their inclination to 3 degrees, eleven inclined planes (the whole number required) have been located, not one of which is straight, and several of which present, in the opinion of the undersigned, curvatures altogether inadmissible. In his report of November, 1829, the undersigned had observed that "the bold outlines of the country did not admit of prescribing to it too many conditions." He will endeavor to show that the condition of having inclined planes in no case exceeding 5 degrees, and in most cases under 3 degrees, which would lead in every case to the substitution of curved planes for the straight planes located by him in 1829, is an unnecessary and inapplicable one, and that such a substitution would be in every way prejudicial to the contemplated improvement. It will be proper, however, to examine the features in which the two lines most widely differ, in the order in which they have been presented, and

First, as to the tunnel:

In the investigation of this subject, and of every other connected with the Allegheny Portage, it seems to the undersigned very clear that we can arrive at no conclusion which has not reference to the amount

and character of the trade to be accommodated on it, or which is not based upon a "consideration of what the exigencies of the trade" may require. If it is doubtful whether the trade in question may be a very small trade or a very large trade, and we must wait for experience to guide us, then, it seems to him, the engineers have hazarded a great deal in recommending a railroad instead of a macadamized road, because it is evident that the latter would be justified by a trade much smaller than would be required to make the former a judicious scheme. But if, on the other hand, the trade on the railroad will, almost as soon as the work is brought into successful operation, be very great, if it will at all approach what has been anticipated by the gentlemen of this State who have given most attention to the subject of its improvements, then it seems equally clear that an increased expenditure ought not in the first instance to be spared, which is to diminish forever afterwards in a much greater ratio than the interest on that expenditure, the annual charges on transportation.

If any inference is deducible from premises, if any conclusion can be predicated on circumstances, analogies or probabilities, it would seem to be safe to predict that a line of railroad would have nearly as large an amount of trade as it could accommodate, which is to unite all the lines of inland communication executed or projected in Pennsylvania east of the Allegheny Mountain, with all those west of it, which must form a link in any combination embracing an eastern and western railroad or canal which is to connect Philadelphia with Pittsburg and the great Mississippi valley and our inland lakes with the ocean. If it should not, that it could result only from mistakes in plan or execution, or a misjudged economy in not executing the work on which so many other lines must depend for their usefulness in a manner commensurate with the purposes it was intended to subserve. The following considerations (presuming this view of the business to be accommodated on the proposed railroad to be correct), appear to the undersigned conclusive in favor of a tunnel:

1st. All the examinations which have been so far made, show a tunnel to be necessary to the location of straight inclined planes immediately east and west of the summit. A reference to the map presented by the undersigned in 1829 will illustrate this position, which, it is presumed, will not be disputed, and a consideration of the great importance of the object in view is reserved to a later period in this report.

2d. In depressing the summit level by a tunnel, we place it below an extensive coal-field, more conveniently situated in this case than any other for the supply of bituminous coal to the towns and villages on the Juniata. In the event of such a work being executed, it is confidently believed that a coal trade sufficient to give a full preponderance to the trade going east, and to lift up that going west, will be created, and that steam engines east of the summit may be altogether dispensed with. On the plan of Col. Long (unless a much more expensive power—that of horses—is substituted), six steam engines would be required east of the summit in addition to those wanted west of it.

3d. The elevation surmounted on the Allegheny Mountain, even after reducing the summit by a tunnel, is one-fourth greater than has yet been encountered on any similar improvement, and the difficulty of accommodating a large trade on a railroad of lifts and levels (superior as such an improvement certainly is to any other plan of overcoming elevations), increases evidently in a much greater ratio than in proportion to the elevation surmounted. Ordinary considerations of prudence would seem under these circumstances to dictate to us, when going beyond what has yet been effected, at least not to go further than may be necessary to our purpose.

4th. In discarding a tunnel, we are increasing the distance between Philadelphia and Pittsburg, or increasing the length of the railroad, so far as time and power are elements in estimating it, about 5 miles, and its disadvantages, if not to an equal, at least to a considerable extent.

The above remark is founded on the presumption that friction on a level line of railroad may be estimated at 1 in 200; of course, that 26 feet and $\frac{4}{10}$ of elevation would require the same power to overcome it as an increase of 1 mile in distance; or that 131 feet of elevation, the additional height of the summit on the portage (should a deep cut be substituted for a tunnel) would be equivalent to an increase of distance of 5 miles.

It is true that the disadvantages of a summit, or of increased elevation on a railroad, are not increased in a ratio correspondent with the increase of power necessary to overcome it, because stationary power, which will generally be resorted to at summits, will be found in most cases cheaper than locomotives or horses, the most expedient and appropriate powers on levels; yet the cost of rope and wear and tear of

machinery will be found in all cases to constitute very large deductions from any superior economy in power, and under certain circumstances may be equivalent to them.

The undersigned trusts enough has been said to satisfy the Board that in recommending a reduction of the Allegheny summit by tunneling, he submitted to them no hastily-conceived or ill-considered project. He has deemed it due to the Board and to himself, whatever may be the fate of the measure recommended by him, to show that it is professionally correct. The Board are too well acquainted with the duties and responsibilities of a civil engineer to expect or desire that their officers in forming their plans should look to their plausibility or popularity. They will expect of them to recommend such as they believe to be most eligible, and to leave the proper authority to adopt or reject.

The undersigned will add that he was very well aware, in submitting in 1829 to the Board of Canal Commissioners a plan of crossing the Allegheny Mountain by means of a system of inclined planes and stationary power, and of reducing the summit by a tunnel, that such a plan must necessarily experience distrust and opposition. It would have been singular, indeed, if it had not. The system recommended was new in all its features in Pennsylvania, and the first essays in the way of tunneling in this State had, from a want of experience on the part of the undertakers, and from other causes, been unnecessarily expensive. Whilst, therefore, the undersigned experiences some regret, he is not surprised that he could not present his views to the Board in a light as forcible as he would have wished. Since the date of his report much has occurred to familiarize the public with the subject of railroads generally, and the complete success of the Carbondale Railroad has enabled the Board of Canal Commissioners to point to an improvement, which illustrates more forcibly than any reasoning could, the advantages of the system proposed for the Allegheny Mountain. The superiority of such a system, whatever may have been thought of it twelve months since, it is presumed is no longer questioned. However bold the assertion may be deemed, the undersigned entertains no doubt that five years hence, when the serious disadvantages of unnecessary elevation, of an unnecessary multiplication of engines, and of curvatures in inclined planes on railroads, are better understood than they appear at present to be, the expediency of tunneling at the portage summit will be questioned as little.

It now becomes the duty of the undersigned to show that on the principles of Col. Long, as exposed in a published document on a former occasion, the expediency of tunneling at the Allegheny summit is recommended by the consideration of reduction in elevation alone. The undersigned is far from concurring in all the views taken in the document referred to. But they will probably be deemed good authority as regards Col. Long himself, when in the present instance that gentleman questions the expediency of a tunnel.

The undersigned here takes leave to refer to a report of the Board of Engineers of the Baltimore and Ohio Railroad Company, made on the 20th of May, 1829, signed by S. H. Long, President of the Board.

In that document, the object of which is to present the result of inquiries made by the Board of Engineers, "in reference to the selection of a route for the railroad from Parr's Spring Ridge to the Point of Rocks," the following views are taken:

"In accordance with an opinion advanced by this Board on a former occasion, in reference to the inclination required for the spontaneous descent of carriages upon a railroad constructed in the manner contemplated for the Baltimore and Ohio Railroad, the cost of transportation upward, on an ascent of 30 feet, perpendicular height, whatever may be the distance; or, in other words, the amount of power required to ascend an elevation of 30 feet, in addition to that required on a level railroad of the same extent, would at least be equivalent to that required for conveyance through a distance of 1 mile. Subsequent experiments on the Baltimore and Ohio Railroad have contributed not only to establish the correctness of the opinion, but also to afford unequivocal testimony that an elevation considerably less than that just mentioned is to be regarded as an impediment more difficult to be overcome than the distance of a mile on a horizontal road. Nevertheless, we shall, on the present occasion, adhere to our former mode of estimating the disadvantages attendant upon ascents, and regard an elevation of 30 feet as equivalent to 1 mile, so far as it relates to the cost of transportation. Agreeably to this assumption, the following table," etc.

It will be unnecessary to give at length the tables which follow. Among the conclusions deduced is this, that on the tonnage anticipated for the Baltimore and Ohio Railroad, the annual difference in the cost of transportation corresponding to a difference in equated distances of 8 miles and 1 165 yards, will be $77 950.

The above views are submitted without comment. The trade on the Allegheny Portage may certainly—without hazard of overrating it—be expected to be equal to that on the Baltimore and Ohio Railroad; and supposing the conclusions of Col. Long to have been even approximately correct, it would be most misjudged economy to dispense with a tunnel, were no expensive substitutes in question, and even if its whole cost could be saved.

The undersigned would now leave the subject of the tunnel, but opinions so exaggerated of its cost and difficulty have been entertained, and so generally, that he is induced to trespass on the indulgence of the Board with a few very simple comparisons. If they produce in the present instance no practical result, they may, perhaps, be useful in inducing a juster appreciation of similar undertakings hereafter.

The whole estimated cost of the Allegheny Portage tunnel was $165 571.20. This amount, like all the items of his estimate, was deemed liberal at the time; but the completion of the Liverpool tunnel, a work of much greater magnitude and difficulty, for a smaller sum, shows it to have been much more than ample.

The height of the Liverpool tunnel is 16 feet; its breadth, 22 feet; and its length, 2 200 yards.

Its excavation, of course, must have been 39½ yards per lineal yard, and the whole cubic contents 86 044 yards.

The whole amount of excavation of the proposed Allegheny Portage tunnel is 50 688 yards.

The cost of the Liverpool tunnel, exclusive of compensation to individuals for houses destroyed, and other damages, was £34 770 sterling, or, estimating the dollars at 4s. 6d., $154 533.

And the Allegheny Portage tunnel, estimated at a proportionate rate, would be $91 034.

In speaking of the Liverpool tunnel, Mr. Booth, the Treasurer of the Company, observes: "But while some portions of the tunnel were excavated under circumstances of no little difficulty and danger, and requiring all the skill and energy of the engineer to accomplish, other portions were hewn through a fine red sandstone, clean, dry and substantial, and requiring neither props or artificial arching; the natural rock forming the roof of the excavation."

It was deemed by the undersigned proper, on account of the horizontal stratification of the Allegheny, to include in his estimate the

expense of arching the whole length of the tunnel; but it is needless to say to any one conversant with the geology of the Allegheny, that a tunnel there would be excavated through a sandstone as clean, dry and substantial as any similar rock, and the excavation of which would, of course, be as favorable as that of the most favorable portion of the tunnel at Liverpool.

The undersigned will now request the attention of the Board of Canal Commissioners to the second point in which the lines traced by himself in 1829, and by Lieut.-Col. Long during the past season, most materially differ, viz., the character of the inclined planes traced on them. A mistake on this head will certainly be deemed far more serious than one in relation to the tunnel. If the tunnel should, for the present, be dispensed with, it may, when found necessary at a later period, be executed, and the best arrangement be then adopted, in relation to the planes immediately east and west of the summit; but a mistake in the location of the planes generally on the line, as it would influence the level and graduation of intermediate portions of it, might be deemed one of irreparable evil. The undersigned begs leave, therefore, to go into this subject at some length, and to introduce the following extract from his report of 1829:

"Two points have been deemed to be of essential importance in the location of inclined planes: 1st—That they should be straight, and that the line of the railroad should, for a short distance from the head and foot of each plane, be in the same direction with the plane; and 2dly—That the plane itself should either have an uniform inclination with the horizon, or an inclination gradually diminishing between the head and foot of the plane. On a plane any extent curved in its plan, a greater loss of power, and wear and tear of cable or rope, will always be sustained than on a straight plane, and without a stage of some extent at the head of the plane, in the same direction with the plane, a very extensive business cannot be accommodated; and should the plane be so far curved as to prevent the manager at the head of the plane from embracing its whole extent under his view, the operation of raising and lowering trains must be conducted by signals, and often disadvantageously.

"On planes on which power is to be applied, an inclination slightly diminishing towards the foot of the plane is most desirable; on self-acting planes (where the fullest useful effect of the gravity of descending

bodies is desired), the most advisable profile is an approximation to the cycloid. It is in either case a disadvantageous feature, if the plane becomes steeper towards its foot. In the one case the engine, at the commencement of its labor, is compelled to exert a greater force than is afterwards required. In the other, a descending train of wagons may acquire, in consequence of its increased stress, combined with its accelerating force, a momentum not to be controlled by a brake, and attended with much hazard.

"It was farther an object, in the case under consideration, to reduce the number of planes as far as practicable. Such a reduction was peculiarly recommended by the saving of time in transportation which it insured, and the superior simplicity of any system of transportation which might be adopted on the completion of the work."

Agreeably to the above views, nine out of ten planes were located, varying in their inclination from $3\frac{1}{2}$ to $9\frac{1}{2}$ degrees; and in one case, in order to command the whole water power of the Conemaugh, at a point midway the plane, an ascent of 26 degrees was admitted. In one of the planes only was any curvature allowed in the plane; and in that case the extent of the curvature was an arc corresponding to a versed sine of 40 feet on a chord of 672 feet. All these planes, it will be observed, have an acclivity greater than 3 degrees, the limit which Colonel Long deems most desirable. It will be proper, therefore, to ascertain whether any very strong considerations forbid such an increase of inclination.

It will, perhaps, on the first view, strike a superficial thinker that the power of the engine required for a plane must be increased in some degree in proportion to its acclivity. It will be perceived, however, at once, by a mind ever so little versed in the simplest elements of mechanics or machinery, that this is not the case; that to lift a weight a given height in a given time, the principle of the inclined plane requires an equal expenditure of power whether it be steep or gentle; and, in practice, something more on a gentle plane, because the same amount of power must in each case be expended in overcoming the gravity of the load, while its friction on a gentle plane is, of course, in the ratio of its increased length. It will equally present itself that in increasing the acclivity of a plane the whole weight of rope will not be increased, because, presuming the plane to be twice as steep, and the stress on the rope, of course, to be doubled, the rope will, on the other

hand, be but half as long; and the strength of rope, it is well known, increases in proportion to its cross-section, or to its weight in a given distance.

It will be equally apparent on examination that the whole weight of the drum, or rope roll, and of the sheaves of the plane, will not at any rate be increased by an increase of acclivity.

If the theory of the inclined plane furnishes nothing to justify the limit of acclivity assumed by Lieutenant-Colonel Long, neither does the practice of British engineers. It is true that in England proper, one of the most champaign countries of Europe, the inclined planes of railroads are generally of but slight inclination. The elevations to be surmounted in that country are not often considerable, and it would, in most cases, have been difficult to have thrown a large amount of rise or fall into a small distance. Self-acting planes have been made gentle for an additional reason, because, as observes Mr. Wood in his treatise on railroads, "Where gravity is the moving power, it scarcely need be stated that the strictest regard should be paid to economizing its effects. The power itself is acquired at no cost, and on that account should be extended to the utmost limit of its applicability."

But in that portion of the interior of England, in the neighborhood of the Peak, which presents an aspect approximating in rudeness and boldness of outline to our own Allegheny, and in the mountains of Wales, the practice of British engineers is very different; and it is believed that the assertion may safely be hazarded, that not a single instance can be cited in those districts in which, under similar circumstances, planes of the gentle inclination proposed by Lieutenant-Colonel Long have ever been executed or proposed.

No improvement in the world presents so striking an analogy to the proposed Allegheny Portage as the Cromford and High Peak Railway. It connects two lines of canal, the Peak Forest and Cromford. Its summit is elevated 992 feet above the surface water of the latter, and the country over which it passes presents obstacles of a similar character. It seems to furnish us, then, a case in point, where the practice of those who have preceded us may lend us some aid. It may be added that the work presents as a study the farther recommendation that it has been but lately executed, and, of course, those who projected and executed it have had the advantage of all the recent improvements in railroads, and that it was planned and executed by Mr. Jessup, one of

the most distinguished among the many eminent civil engineers of Great Britain.

In the "London Year Book" for the year 1830, the following statement is given of the inclined planes on that work:

1st	ascent	1 740	feet	long	204	feet	1–8.53	rise.
2d	"	2 133	"		261	"	1–8.17	"
3d	"	2 124	"		253	"	1-8.40	"
4th	"	1 371	"		98	"	1–14	"
5th	"	2 550	"		168	"	1–15.2	"
6th	descent	1 980	"		266	"	1–7.44	"
7th	"	1 365	"		191	"	1–7.15	"

It will be seen on examining the above inclinations that they vary between 3¾ and 8 degrees in surmounting a summit of 992 feet, and that the elevations overcome by the inclined planes vary between 98 and 266 feet; evidently showing that the inclination as well as the extent of the planes was deemed a matter of no particular moment, and was governed by localities, and that the peculiar advantages of a plane not exceeding 3 degrees of inclination were, in a case similar to that under consideration, not perceived by Mr. Jessup.

In Wales the undersigned scarcely recollects an inclined plane where under 10 degrees, and those that he saw in the mining districts of that province varied generally between 10 and 30 degrees; and he is informed by Captain M'Neil that he saw one on the Blaencum Railroad, near Abergavenny, as steep as 35 degrees.

It is scarcely worth while to add anything farther on this subject. It will be seen on reference to the profiles submitted in 1829, that the range of inclination proposed by the undersigned on the Allegheny Portage, with one exception, does not vary materially from that adopted by Mr. Jessup on the Cromford and High Peak Railway; and that, in the case in question, an important object, that of securing a water power equal to the accommodation of any trade which can ever be anticipated on the portage, seems to recommend an increase of the angle of inclination of the plane. It will be agreed that this particular plane is inconveniently steep, and that but for the circumstance above stated a different location would have been deemed more advisable; but it seems also equally clear that, where a great elevation is to be surmounted, planes, even if straight, may be inconveniently gentle, because

they ought not to be more numerous than necessary, and, in the language of Mr. Sylvester, "the length of an inclined plane ought never to be such as will prevent a person from seeing the whole course of it from top to bottom."

The undersigned will now proceed to examine the character of the planes proposed by Colonel Long as substitutes for those to which attention has been drawn above, and he will premise that, however objectionable he thinks he can show them to be, he deems their defects as necessarily resulting from the inapplicability of the system and principles on which they were traced. In a rugged country like that over which the proposed connection is contemplated, inclined planes must be located on the slopes and spurs of mountains often intersected with wide and deep ravines. It, under these circumstances, may be deemed next to impossible to trace a line, descending at any given inclination, for a very considerable distance, without changing its direction: and, therefore, planes of gentle inclination must necessarily be either more numerous than would be advisable, or curved. Satisfied, from his knowledge of the country, that the principles adopted and limits of inclination preferred by Lieutenant-Colonel Long would necessarily lead to this dilemma, and believing that an unnecessary multiplication of planes, or curved planes, were evils equally to be deprecated on a line of railroad as important as the one contemplated, the undersigned was induced to test the correctness of his views by a comparison with those of two other gentlemen distinguished for scientific and practical attainments in the profession. The engineers alluded to were Mr. John B. Jervis, Engineer of the Carbondale and Albany and Schenectady Railroads, and Horatio Allen, Esq., Engineer of the Charleston and Hamburg Railroad, who, it will be recollected, was deputed to England, by the Hudson and Delaware Coal Company, to obtain information in relation to railroads, and fixed and locomotive machinery generally. The following are extracts from the letters of these gentlemen in reply to his inquiries:

Mr. Jervis observes, speaking of the Carbondale Railroad, "our inclined planes are all straight. I should deem it a great objection to have such a curve as would prevent the men from seeing the operation from end to end. A moderate curve may be admitted: but I should want a strong inducement to admit any essential curving on a plane that had heavy work to perform. I should vary the inclination very

much before I would curve for a road that had extensive business, or incur more expense in construction."

Mr. Allen, in a letter of the 22d of January, says: "I would decidedly prefer increasing the acclivity very considerably before I should admit a change of direction in the plane, and when following this principle would lead to extreme cases of ascent, I think I should increase the number of planes rather than introduce curvatures. The sources of derangement and accident are sufficiently numerous in inclined planes without adding those which are inseparable from curves."

The annexed volume, marked P, exhibits protractions of the planes traced by Colonel Long, and proposed by him as substitutes for the straight planes which have been the subject of examination. The undersigned confesses that he feels some reluctance at going into an argument to prove the injudiciousness of adopting such planes on such a work as the Allegheny Portage. He has no hesitation in saying that on those of them which present the largest amount of curved line an endless rope or chain (the machinery which, except under particular circumstances, should be preferred on a plane of uniform inclination), if it could be worked at all, would be cut to pieces almost immediately; and he thinks it would not be difficult to show that the waste of power, and wear and tear of machinery on any plan of working which might be proposed, would be immensely increased.

Colonel Long has not, however, submitted to the Board of Canal Commissioners his views as to the manner of transportation on the proposed railroad, and his estimate, it will be observed, embraces no item for machinery of any kind. The undersigned is, therefore, without a fact to indicate to him the plan of working which Colonel Long proposes for his planes. Under these circumstances, he begs leave to submit the following views in illustration of the position he has taken, the force of which he thinks will be appreciated by the Board and admitted by scientific individuals who may chance to peruse them.

The plan of applying stationary power on planes which is attended with the least friction of rope, is that which purposes attaching the load to the end of the rope on the reciprocating system. Such a plan of working would be in general unadvisable on planes having a uniform angle of ascent, because, as must be obvious, the engine has, on this plan, not only to raise its load, but the weight of the rope to which it is attached, and which, on any plan of endless machinery, would be coun-

terbalanced by the extension of the rope along the opposite descending track. The increased friction of ropes by curves will, however, be considered on the plan first suggested, because on that plan it is smallest.

Now, on a straight plane of any length or elevation, the friction on the rollers is evidently in proportion to the weight of rope, and on any one roller is as the rope resting on that roller, or as R.

But if the plane be curved, then the stress on each roller is made up not only of the weight of the rope, but of the resultant of two forces, viz., the stress of the load (including the stress and friction of the portion of rope below the roller), and at least an equal force required to overcome this stress; or supposing

s = the stress of the load and rope below any given roller.

D = the distance between the rollers on the plane; and

r = the versed sine of a portion of the arc between any two alternate rollers.

Then the pressure on any roller on a curved plane $P = R + \frac{2rs}{D}$

Applying this formula, and selecting from "Wood on Railroads," as the subject of investigation, the plane which approximates most nearly to the standard of limitation (3 degrees) preferred by Lieutenant-Colonel Long, we have a straight plane 2 325 feet long, with a rise of 115 feet, on which a rope 7¼ inches in circumference is made use of to raise seven loaded carriages, weighing 9 408 pounds each. The rollers on the plane appear to have been about 20 feet apart.

Now, the pressure on each roller of the above plane, or the quantity R of the above formula, would be simply the weight of 20 pounds of 2¼-inch rope, or about 18 pounds.

Let us suppose a plane precisely similar to the above, except that it should be curved in arcs of 600 feet radius. Presuming the load to be precisely what it is stated in Wood.

The stress on the rope, comprising both the gravity and friction of the load, and estimating this last at 1 in 100, in consequence of the curvature of the plane, would be, at a point near its foot = $\frac{9\,408 \times 7 \times 115}{2\,325} + \frac{9\,408 \times 7}{150}$ or 3 697 pounds; and the versed sine of 40 feet in length of an arc of 600 feet radius is $\frac{3.3}{10}$ of a foot. Then, substituting the symbols in the above formula, we should have for the stress on a roller at the foot of the plane $18 + \frac{3\,697 \times .33 \times 2}{20} = 18 + 121$

=139 pounds. Consequently the ratio of pressure, or of friction, on the friction rollers or sheaves of the above planes, near the foot of the planes, is as 18 to 139, or as 1 to $7\frac{7}{9}$.

But it is evident that the pressure on each roller, commencing at the foot and advancing to the head of a plane, between the foot and head of a curved plane, increases rapidly, because we have to add both the stress and friction of rope below each roller to the stress and friction of the load in ascertaining (the resultant which gives us) the pressure on the roller; and it is to be recollected that in estimating this stress and friction we are to estimate for a rope of larger dimensions than would be required on a straight plane, as well on account of the increased strain of the rope as its increased wear and tear. It is not, then, hazarding too much to say that the increased pressure in cases above supposed would be at least ten-fold on the curved plane.

Now, the friction of rope on the sheaves of a plane is, of course, proportionable to the pressure, and, whilst no rule can be laid down for determining the ratio of the wear and tear of rope to the friction on the sheaves (because a part of this wear and tear is experienced at the rope rolls, and results also from the rigidity of cordage and strain of the load), we know that it depends very materially on it. The following considerations show that we ought to be very wary of increasing (as we should in a formidable ratio by curvatures in inclined planes) a charge on transportation under any circumstances considerable.

On the Liverpool and Manchester Railroad, Mr. Booth, in his late pamphlet on that railroad, observes that "the curves seldom exceed a deviation from a straight line of more than 4 inches in 22 yards, forming a segment of a circle which, if extended, would embrace a circumference of 15 miles." Nevertheless, in an estimate of the annual cost of transportation on that road by stationary power, made by Messrs. Walker and Rasstrick, in 1829, the annual wear and tear of ropes on the roadway is estimated by the former gentleman at upwards of £11 000 sterling, or one-third of the whole expense of transportation, whilst by the last named engineer the whole cost of transportation was estimated at £33 317, and the wear and tear of ropes alone at £14 820, a sum nearly equal to five-sixths of all other charges on transportation.

On the Rainhill and Sutton inclined planes, the annual expense of ropes alone was estimated by Mr. Walker at £3 315, or at twice the amount of all other charges on transportation on those planes.

In our own country, the wear and tear of rope has been found by Mr. Josiah White, on a straight plane, to be equal to one and one-fifth cents per ton in descending 215 feet.

To these statements the undersigned begs leave to add the following extract from the report of a deputation of the engineers of the Liverpool and Manchester Railroad, appointed to visit the railroads in the North of England, made at Liverpool, October 22, 1828.

"From Newcastle the deputation went to Sunderland, and visited that end of the Hetton Railway which terminates at the shipping place on the River Wear. The quantity of coal brought along this railway was stated to be from 12 to 1 500 tons. The rope was considerably thicker than on Mr. Thompson's railway, and was stated by one of the engineers to be worn out in 4 or 5 months. The plane next the shipping place is 1 968 yards long, worked by a forty-horse engine. The curves near the port are very abrupt, and must necessarily very much increase the wear and tear of the rope on that part of the line."

It is believed to be unnecessary to expose further the serious evils which would arise on an important line of railroad from curvatures in inclined planes, to the extent proposed by Lieut.-Colonel Long.

It will be observed that the undersigned has so far avoided any investigation of the details of the line submitted by Colonel Long. He is willing to presume, if the outlines of his plan of improvement are judicious, its details are so likewise. One of the suggestions of Colonel Long, in particular, the undersigned has certainly no objection to. This is the introduction of a tunnel at one of the bends of the Conemaugh, by which the length of a line along the valley of that stream may be reduced something more than a mile. He will unite with pleasure with Colonel Long in submitting this suggestion to the careful consideration of the engineer who may execute the work, believing the object in view ought certainly to be attained, if the changes in the line necessary to effect it would not in other respects impair it, and if it can be effected without introducing a curved plane, which on the line traced by Colonel Long appears to result from it.

Presuming the Conemaugh tunnel proposed by Colonel Long to be incorporated into his plan, the length of the line proposed by the undersigned would be about thirty-seven miles, instead of thirty-eight miles and fifty-three poles, as stated in his report of 1829. It will be observed by the Canal Commissioners that the line of Lieut.-Colonel

Long is stated at but about 35⅞ miles. The causes of this apparent difference between the two lines (for as they cross at the same summit, and are traced along the same valleys, and often alongside of each other, it is obvious it can only be apparent), the Board will at once know how to account for by recollecting:

First.—That the length of the line traced by the undersigned in 1829 is computed to the head of the basin at Hollidaysburgh; whereas that of Colonel Long is terminated at the Hollidaysburgh bridge, a point about three-eighths of a mile short of the basin.

Second.—The line of Colonel Long has been traced in regular curves and tangents, an operation which always results in a reduction of distance, but which, as it is a work merely of routine, and one of labor and expense, the undersigned is not in the habit of performing until a line is definitely determined on, and a work about to be executed.

The undersigned ought not to close this report without adverting to the difference in the estimates of Colonel Long and himself. The whole cost of the line surveyed by that gentleman, it appears, is but $611 700, whilst that of the undersigned, including the tunnel at the summit, is $936 000. The commissioners will not fail to remark that Colonel Long has made no estimate for steam engines or machinery of any kind; that nothing is included in his estimates for superintendence or contingencies; that they extend only to a point near the Hollidaysburgh bridge, and that they do not include the lengths of railroad which must be made parallel to each basin. Passing by these points, to which the undersigned respectfully requests the attention of the Board, he begs leave to submit the following remarks with regard to the aggregate allowed by Colonel Long for roadway formation.

This item of expense, as the Board well know, may be more or less; and it is among the most delicate of the duties of an engineer, and the one which requires the exercise of his best judgment in contemplating an expenditure for this purpose, to have constantly in view the objects to be effected by an improvement. If the trade on a railroad is to be a small one, it is evidently injudicious to expend too large an amount for a more perfect accommodation of it. If it is to be a large trade, then sound economy will dictate such an expenditure in preparing the road bed as will give to the power employed on the railroad its fullest useful effect, and reduce, of course, in the largest practicable proportion, the expenses of transportation.

Believing, as the undersigned did, that the Allegheny Portage Railroad would, if properly executed, have a larger trade than any similar work in this country, he did not, in choosing a line, select that which would be most easily formed into the road-bed of a railroad, but that which seemed to him to possess the greatest capabilities, or on which a railroad, when executed, would be most valuable. He felt that a very small reduction on the charges of transportation would be more than equivalent to the interest of a large additional expenditure in first cost, and that it would be most injudicious economy, for the sake of even a considerable saving in the preparation of the road-bed, to tax forever afterwards the trade on the road unnecessarily. He confesses he should have deemed it as a great mistake to have feared a liberal expenditure where the object to be effected by it was so important as to incur an extravagant one on a railroad destined for the accommodation of a small traffic.

The whole cost of roadway formation for the Allegheny Portage Railroad, according to the line and estimate of Lieut.-Colonel Long, will, it appears, be "one hundred and eighty-two thousand one hundred and seven dollars and eighty cents, which gives for the average expense per mile, inclusive of a tunnel, one thousand feet long, five thousand and eighty-six dollars and fifty-nine cents."

The undersigned would respectfully submit a single fact to the Board.

He is executing in the anthracite district of this State, a railroad along the valley of a stream (the Little Schuylkill) highly favorable to a work of the best description. As the object of the improvement was a special one (the accommodation of a coal trade), it will be conceived that, though it was deemed advisable to execute the work in a substantial manner, no unnecessary expenditure would be incurred on it. The cost of the roadway of this railroad (20½ miles), executed under uncommonly favorable circumstances, exclusive of the wood-work of bridges, will be about one hundred and five thousand dollars, or a larger average per mile than is allowed by Colonel Long for the roadway formation of a railroad, including bridges, which is to cross the Allegheny Mountain.

The undersigned would remark, in closing this report, that in accepting the appointment which the Board did him the honor to confer on him, he felt scarcely disposed to believe that any facts or views which might be developed on farther examination would induce a change of the opinions expressed in his report of the 21st of November, 1829. He

should, indeed, have performed his duty to the Board but indifferently, and should have deserved very little their confidence, had his plans been formed on such slight data as to induce a belief that they would be materially changed on further examinations of himself or of others. He was happy, however, to meet the wishes of the Board, and to have an opportunity of considering any suggestions on the part of the gentlemen with whom he was associated, and he has no doubt the Canal Commissioners will do him the justice to believe that if any other plan for crossing the Allegheny had, on reflection, appeared to him preferable to that submitted by him in 1829, he would have had the candor to acknowledge it.

He regrets that the opinions of Lieut.-Colonel Long and himself differ so widely. That gentleman has, however, believed it his duty to offer, as a substitute for the line surveyed in 1829, one, the merits of which the undersigned has certainly not perceived. He has had, under these circumstances, no alternative but to present the above investigation. This duty to the Board and to himself being performed, the undersigned now takes leave of the subject.

All which is respectfully submitted.

MONCURE ROBINSON,

Civil Engineer.

PHILADELPHIA, March 5, 1831.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XV.—April, 1886.)

DISCUSSION ON

THE SOUTH PASS JETTIES.—TEN YEARS' PRACTICAL TEACHINGS IN RIVER AND HARBOR HYDRAULICS.*

DISCUSSION BY COL. W. E. MERRILL, D. J. WHITTEMORE, GEN. C. B. COMSTOCK, A. P. BOLLER, CAPT. J. C. POST, E. P. NORTH, R. E. MCMATH, A. C. SAVAGE, O. CHANUTE, CAPT. W. H. BIXBY, J. F. LEBARON, GEN. Q. A. GILLMORE, E. L. CORTHELL AND JAMES B. EADS.

Col. WILLIAM E. MERRILL, M. Am. Soc. C. E. —The paper on the South Pass Jetties, by Mr. E. L. Corthell, M. Am. Soc. C. E., is a valuable and interesting one, but there are some errors in it which are worth noting, and it contains certain suggestions that should not be permitted to pass without comment.

The South Pass Jetties have given an excellent mouth to the Mississippi River, and their success has been such as to warrant the United States in paying for them a large bonus over the cost of construction; but it should not be forgotten that the engineer who built them contracted with Congress to give a depth of 30 feet for a width of 350 feet for a total sum of $7 250 000, on the principle, widely advertised, of "no cure, no pay," and failed to secure the depth. As a matter of fact, he

* The South Pass Jetties.—Ten Years' Practical Teachings in River and Harbor Hydraulics. By E. L Corthell, M. Am. Soc. C. E. Transactions Am. Soc. C. E., No. 290, Vol. XIII, October, 1884.

only earned under his contract the first three installments of half a million each, with a quarter of a million for maintenance, making in all only $1 750 000. The fourth installment of half a million was not payable until a channel had been secured having a depth of 26 feet with a width of 300 feet, and no such channel has even yet been secured. In order to be sure of the exact facts in the case, I, to-day (January 21st, 1885) telegraphed to the inspecting officer to know the exact width of the 26 foot channel, and have just received the reply that it is 210 feet wide.

In March, 1879, Mr. Eads procured the passage of an Act of Congress reducing the required width of the 26-foot channel from 300 to 200 feet, abolishing the clauses requiring the 28-foot and 30-foot channels, and redistributing the payments, so that he should receive the same sum ($7 250 000) for the 26-foot channel as he was to have received for the 30-foot channel. It is true that a provision was inserted requiring a central depth of 30 feet regardless of width, but this clause added nothing to his obligations, as it was hardly possible to secure the required depth of 26 feet over a width of 200 feet without having a central depth at least 4 feet greater. His original contract for a channel 30 feet deep, with a width of 350 feet, would probably have carried with it a central depth of 35 feet.

I mention these facts not in the way of criticism, or personal allusion, but merely to show the great difficulty of making results tally with predictions when dealing with large bodies of water in motion, and to show that iron-clad contracts with the Government can sometimes be set aside. In my judgment there was much more engineering skill and fertility of resource shown in the work at the head of the South Pass than at the jetties themselves. There were abundant precedents for the latter, but none that I know of for the former.

The lessons which Mr. Corthell draws are nine in number, and on them I would like to make a few comments.

In his condemnation of the "outlet" theory, as applied to the Mississippi River, I fully agree, but my acquaintance comes from a study of the river above New Orleans, and I do not see that experience at the jetties has any particular bearing on the subject. No engineer has ever disputed the fact that by contracting a stream you can deepen it; in fact it is an elementary principle, and it has been applied on the Ohio River with moderate success since 1825. I say "moderate" success, because local deepening is often followed either by the formation of a new bar below the contraction, or by a decrease of depth on the bar next above.

I do not think that the criticisms on the work at Galveston and at Charleston are justified by the facts. In building jetties upon movable sands the lower courses must necessarily be built first, and from the nature of their construction they can subsequently be widened or

heightened as occasion may demand, without making the ultimate cost of the jetty materially greater than if the intention to build it above the sea level had been proclaimed from the beginning. Nothing has been done at either locality, except what would have had to be done in any event.

It seems to me that a wise and prudent engineer, dealing with such a difficult problem as the improvement of a harbor on a sandy shore, would proceed cautiously, feeling his way by degrees, prepared to change his plans whenever their effect seemed unfavorable. I am disposed to believe that, as a rule, jetties should be above the level of the sea, as they are at Ymuiden, the mouth of the Maas, Port Said, and many other localities; but in spite of this belief I should certainly try first to build two submerged jetties, and see how they worked, because it is quite possible that one or the other might require removal. It is also possible that, in some localities, submerged jetties may answer every purpose, at a great saving in cost; in other places jetties above water may confine storm tides too much, and may cause too violent an action within the jetties, and create a shoal bar in the open sea beyond them. To illustrate the necessity of making haste slowly, and the difficult nature of the general problem, I would refer to the sandy harbors along the northeast coast of France and the north coast of Belgium, where jetties have proved a practical failure, even with the aid of large flushing basins and high tides.

In spite of the large experience of the French and Belgian engineers with this class of harbors, the King of the Belgians a few years ago offered a large prize, open to the engineers of all nations, for the best essay on harbors on sandy coasts; the result of the competition I have not learned, but it is clear that the engineers who have had most experience with these harbors consider the problem a very difficult one. The failure at the mouth of the Maas is a sufficient proof that it is not always possible to predict the results of jetties on sandy shores, even though a large river flows out between them.

In reply to the criticisms on Galveston Harbor, contained on page 326, I will submit the following statement from Col. Mansfield, the engineer in charge, to whom I forwarded Mr. Corthell's article, with the request that he would send me a reply.

Col. Mansfield quotes the paragraph beginning on the third line of page 326, and replies as follows:

"This budget of errors, and the errors following, as regards results, may be best answered by giving a brief account of the work at Galveston, so far as executed, with accompanying results, and stating what is intended to be done in the line of further improvement.

"I will, right here, call attention to a most important consideration, which seems to be entirely overlooked, viz., the condition of the two problems—the one, the removal of a bar formed by the detritus from a

great river, where immediately beyond the crest of the bar and the jetty heads is the deep water of the Gulf, ready to receive the material brought down by the current and scoured from the bar, where it can no longer obstruct navigation—the other, the removal of a bar formed by sand drifted along the coast and heaped up by the sea at the entrance to a bay into which no rivers of any size enter.

"There is no parallelism in the case of the South Pass, Mississippi River and Galveston Bar. The problems being so entirely dissimilar, they must be treated separately, and upon a careful study of all the conditions surrounding each. A treatment that would solve the one might not solve the other.

"Galveston Bay has an area of 451 square miles, forming an immense reservoir, whose surface is known to have been raised to a height of 10 feet in a great storm, and not infrequently raised 3, 4, 5, and even 6 feet by easterly winds, though the average range of tides is but $1\frac{1}{0}$ feet. The discharge of accumulated waters of the bay under off-shore winds which usually follow easterly blows, creates strong currents through the gorge, maintaining here 40 feet of water; and the momentum of discharge is sufficient to excavate and maintain a channel of 30 feet to a distance of over a mile, 25 feet a distance of 2 miles, and 20 feet over 3 miles out from the gorge.

"The crest of the bar is about 4 miles out, and the distance across from 18′ to 18′ is about $1\frac{1}{4}$ miles, and beyond the exterior 18′ curve the bottom of the Gulf deepens at the rate of about 1 fathom to the mile.

"Beside the outer bar obstructing the entrance to the bay, there was an inner bar obstructing the entrance to Galveston Channel, along which the wharves are located.

"Upon this inner bar there is now 26 feet of water at low tide, where formerly there was but about 9 feet, and the depth of water over the outer bar has been improved, so that navigation has been benefited to the extent of about 2 feet, and this has been accomplished by the Government works, which consist of an incomplete jetty, running from Fort Point out to the crest of the bar, a distance of $4\frac{1}{4}$ miles.

"The project contemplates the raising of this work to a greater height, if necessary, and the construction of a jetty on the north side, the position of which is yet to be determined.

"Previous to my taking charge, in 1880, Major Howell had expended $527 000, and secured as a result 20 feet of water on the inner bar. I have expended since, in the construction of the jetty, $975 000.

"This structure is a most substantial one, and is not likely to deteriorate much; that it has been an economical one, the records will plainly show.

"Should it be necessary to raise this jetty above high water mark, it can be done for less than $1 000 000, while the jetty upon the other side, equally high, may be built for less than $2 000 000.

"My opinion is that it will not be necessary to raise the jetties to a height above overflow; to be sure, the greatest effect will be produced thereby, but the question, if any, is, will it be necessary in order to secure the required depth of channel? I think not, nor will it be necessary to raise the jetties so high in order to guard the channel against filling from wave action. Let me give you my experience:

"At Aransas Pass Bar, my jetty, nearly a mile in length, originally brought near to the water surface at mean low tide, but since compressed so that there was an average of more than 4 feet over its entire length, caused a channel to cut where desired through the bar, 11 feet deep, where there was but 4½ feet before; and this channel has been maintained a long time and is improving. I am now raising this jetty above overflow.

"Here is a single jetty, not on the side whence the drift comes, but on the other side, and nothing in the way of a jetty to prevent the whole drift of sand along the coast from coming into the channel.

"No two rivers or harbors are precisely alike, and to lay down in advance a law that shall be rigidly applied to all alike, without regard to the varying circumstances, would be the height of absurdity."

The statement of Mr. Corthell, that there is to be 2½ miles between the jetties, is positively denied by the Chief of Engineers and by the Resident Engineer, both of whom state that the north jetty has not yet been located. Mr. Corthell asserts that "lateral outlets were left near the land, ostensibly to let the tide into the bay," and he vigorously condemns this method of construction. I agree with him, but I can find no authority for the existence of these outlets; nothing of the kind appears in the last official map of Galveston Harbor. There is a low place in the jetty, about 3 miles from the shore, but this is nowhere "near the land," and it appears to be merely a place where the work is incomplete. The map shows that the shore-end of the jetty, for at least a mile from shore, is above the water surface.

The total expenditures made by U. S. Engineers on the improvement of the outer bar at Galveston is a little less than $1 000 000. This is just ⅛ of the sum that Mr. Eads asks Congress to appropriate in a lump for his project. I would like to ask the author of the paper if he or any one else could have done any more with the sum allowed by Congress than Col. Mansfield has done, and how he or any one else could have completed an $8 000 000 job for $1 000 000. It is understood that Mr. Eads proposes to build a pair of jetties just as the Government engineers propose. With an $8 000 000 appropriation he can undoubtedly work much faster than a Government officer with $200 000 a year, but the nature of the problem is such, that in any event he must do about the same amount of work in pretty much the same way. The only possible variation is in the location and length of the jetties, and as his location has not been divulged, it cannot be criticised. Experience gained in building the existing south jetty shows that it can be raised above water

level for $1 000 000 more. The north jetty can be built at the same cost as the south jetty, and the entire improvement can be completed for $3 000 000. Why, then, should the United States give any one a bonus of $4 000 000 for doing what can be done just as well by its own officers at cost, and without any bonus at all ?

If the United States is not satisfied to use its own officers to do this work, then I submit that the matter should be thrown open to public competition, and all members of the American Society of Civil Engineers should have an opportunity to submit proposals. It is more than probable that competent engineers and contractors can be found who will guarantee 26 feet of water for a much less profit than $4 000 000. Congress has passed laws requiring that contracts be given to the lowest bidder, and yet we find that Congress itself is seriously discussing the question of selecting one man out of the whole nation, and giving him, without competition, an enormous contract, at more than double the estimated cost of the work.

The practical effect of such a contract is to make an immediate appropriation of nearly $8 000 000 for the improvement of Galveston Harbor. Had less than half this sum been appropriated when work on the outer bar began, the work would now be completed. Mr. Corthell's statement that $1 500 000 have already been expended on Galveston bar (meaning the outer bar) is incorrect, a very large amount having been expended on the inner bar, a very necessary work, but wholly disconnected with the outer bar. The exact expenditure on the south jetty has been $975 000, or less than $1 000 000.

The final lesson that Mr. Corthell enunciates is, that civil engineering work should be in the hands of civil engineers. He seems to forget that United States military engineers are thoroughly educated as civil engineers, and that in many particulars experience in the two branches is identical. Moreover, all the older army officers have had years of experience as civil engineers, and it is as civil engineers that they are admitted into the American Society of Civil Engineers. He makes a plea that "the doors be thrown open freely to all engineers fitted by education and experience for the service required ;" to this I make no objection. If Congress should choose to select Mr. Eads as the engineer of the works at Galveston harbor, I should not raise a word of protest, for I have as high an appreciation as any one of his genius as an engineer ; but when he comes forward as a contractor, in such a contract, the matter stands on a different footing, and I feel it my duty to point out the objections.

In conclusion, I would call Mr. Corthell's attention to his error in asserting that in no other country are military engineers employed on civil works. Great Britain makes exactly the same use of her military engineers as does the United States.

D. J. WHITTEMORE, M. Am. Soc. C. E.—Not being personally familiar with the purely engineering matter treated in Col. Merrill's criticism of Mr. Corthell's paper, I offer no opinion relating thereto. In regard to the final lesson drawn by Mr. Corthell, to which Col. Merrill refers, some remarks by me may be proper.

Our national works of a civil character have, through wise, and sometimes unwise, Congressional legislation, reached proportions of great magnitude, and there can be no question that the charge of these works should be in the hands of the best engineering talent that our country affords. It also must be admitted that this civil work should be in charge of men educated as civil engineers. It is also true that the engineer-officers of the army are educated not only as military engineers, but also in all the branches pertaining to civil engineering—indeed some of our ablest engineers have been and are graduates of West Point—and it is quite natural that our Government should select, from among its agents having scientific culture particularly fitting them for the duties referred to, the engineer-officers of the army. While I believe our Government has generally acted most wisely in these appointments, I still believe that in the near future the civil work of this nature should be in charge of national civil engineers, holding their positions separate and distinct from the military department. To the accomplishment of this end it is my opinion that there should be established a national polytechnic school for the education of civil engineers for the civil service, separate and distinct from the military department, and that all so educated should be as completely enrolled in the service of the Government as are the officers of our army, and that all thus enrolled should be subject to the rules of a just and strict civil service. At the same time it should be possible that any of those members of our profession educated elsewhere, who from an examination before a proper commission could show competency, might enlist in said service on an equal footing with others educated at the national school. In this way it might be hoped that our civil service in matters relating to engineering would at least excel that of England in India, and approach, if not equal, that of France. It should be such a service as would be attractive to our ablest men, not only in civil life, but to military engineers, who from past service, experience and inclination, might prefer such a service.

Throwing the doors wide open for the employment of civil engineers in this branch of Government service, without the safeguard of strict civil service rules, would be fraught with great danger to the honor of our profession. Political influence would too often govern appointments. Rather than have unworthy and also perhaps worthy members of our calling striving for official preferment through political engineering, it would be much better that the work remain in the charge of the engineer officers of the army.

Gen. C. B. Comstock, M. Am. Soc. C. E.—The paper of Mr. Corthell on the South Pass Jetties contains much interesting information. It, however, draws some rigid conclusions from his experience there, that he apparently would apply to jetties between which there is no large fresh-water flow; conclusions in which engineers who have experience in such works will scarcely concur. For instance, after speaking of the full volume of tides, he concludes that "jetties to be of the greatest effect, should be brought above the surface of the water at flood tides." It is stated that this is also necessary to prevent sand from washing over the works, and that "experience shows this to be absolutely necessary to preserve the channel."

Where the water of a river like the Mississippi, at its mouth, is at hand for use in securing a channel between such jetties, the work of improvement is a relatively simple one, namely, to contract the river enough to scour the depths required.

No advantages, save those of economy, result from keeping the jetties low. But when the scour results from the action of tides or winds, other and more complicated elements enter. The scour arises entirely from the filling and emptying of an interior area, and the inflow equals the outflow. To put in works which will seriously interfere with the filling of the interior basin, is to destroy the very agent on which one depends for scour.

When these high jetties would produce such a result, their use is inadmissible. They might also be inadmissible if they produced too great an advance of the shore line.

The more knowledge one has of the improvement of tidal entrances, the less confidence does he feel in positive predictions as to the results projected works will accomplish. Where it is practicable to feel his way, determining as far as possible from experience on the spot the best height to which jetties should be raised in order to obtain the desired channel, to avoid diminution of the interior tidal prism, in narrow channels to give water still enough for navigation, and to prevent large and injurious motion of sand over the jetties into the channel, the skillful engineer will be glad to do so, rather than to tie himself to a preconceived plan and to rigidly carry it out, neglecting the lessons the work itself teaches.

At the mouth of the Adour the jetties are of open work to permit the motion of the sand into the channel, instead of accumulating it against the outside of the jetty and advancing the shore line; at Calais and Boulogne the solid jetty is kept much below high water for the same reason; at the mouth of the Maas the jetties are kept below high water; and the training walls which have so much improved the lower Seine, are kept, in its estuary, nearer low than high water. The height of jetties must then depend on the place.

Each bar has its own peculiarities, and the engineer who attempts to

apply a cast-iron rule to them all will fail. Sir John Hawkshaw, a high authority on harbors, says there is "nothing more certain than that each one must be dealt with according to its own special *regime*."

The writer also concludes "that properly designed jetties with a deep channel and strong outflowing current will not only not accelerate the advance of bars, but will greatly retard their growth."

Before the construction of the South Pass Jetties, the shore line was advancing at the rate of about 100 feet a year. About the same amount of sediment is still going out in each year to be dropped beyond the ends of the jetties. The water issuing from the jetties with higher velocity, carries some of the sediment farther out than before, resulting in a slower advance of the shoal contours, and a more rapid advance of those in deep water. Thus in eight years the 20-foot curve in front of the jetties has advanced 344 and the 90-foot curve 1 018 feet.

Captain Turtle's report in the Report of the Chief of Engineers, 1884, shows that for a trapezoidal area of 1¼ square miles, whose bases are perpendicular to the prolongation of the jetties, one base being 400 feet in front of the jetty ends and 3 500 feet long, the other 5 600 feet in front of the jetties and 9 700 feet long, the mean depth in June, 1884, was 52.9 feet; that the mean fill since 1876 had been 8.29 feet; and that the mean fill from 1883 to 1884 was 3.61 feet. In other words, the average fill per annum of 1.03 feet over this area increased last year to 3.61 feet. If of this trapezoid, a narrow strip 1 000 feet wide, lying in the prolongation of the jetties be taken, its length will be 5 200 feet. For its center portion, having an average depth of 57 feet, the average shoaling for the year 1883–84 was 2.7 feet; for its middle portion, having an average depth of 43.5 feet, the average shoaling for 1883–84 was 4.5 feet; in its inner portion, having an average depth of 26.5 feet, the average shoaling for 1883–84 was 5.5 feet. A part of the great increase in shoaling in 1883–84 over that of preceding years, may be due to the rapid fall of the river in June, 1884.

In eight years the average mean annual advance of curves of equal depth lying in front of the jetties—the curves considered being those of 10 feet apart in depths from 30 to 100 feet inclusive—had been between 70 feet as a minimum and 126 feet as a maximum.

The Government Board of Engineers which first made a plan for the improvement of the South Pass (see Report of the Chief of Engineers, 1875, Part 1, page 945), found the average annual advance of the shore line to be then 100 feet per annum. The figures just given show about the same rate of bar advance as before the work was done, and give no support to the belief that jetties will greatly retard the permanent advance of sedimentary bars, nor to the belief that the jetties without prolongation will maintain a 30-foot channel to the sea during either our own time or that of our posterity.

An erroneous impression is conveyed in saying that Gen. Barnard,

in distinction from the other United States Army engineers, was an earnest advocate for the application of jetties to the mouth of the Mississippi River. This statement has often been made.

The facts are that Congress, in 1874, ordered a Board of Engineers; that this Board, after extended investigation, made the first plan for the improvement of the South Pass by jetties, and recommended its adoption. There were three officers of engineers on the Board, and two of them were strongly in favor of jetties.

It seems, then, that of the few officers of engineers officially connected with the plans for improving the mouth of the Mississippi, three were in favor of jetties.

The writer also concludes that Government "should not give the general or exclusive charge of public improvements into the hands of engineers educated to conduct works of a totally different character." As army engineers for the last twenty years have had little to do except construct river and harbor works, it is difficult to see how a better education for the purpose could be given.

All leading nations have a body of permanent Government engineers on whom they depend for the execution of their Government public works. Experience has shown them that only by having permanent servants can they be faithfully served. The United States, with less system and permanence in all its administrative arrangements, has all the more need of permanent servants in the expenditure of public money. Granted, then, that a body of Government engineers is a necessity, the question is: Having such a body already organized, with little other employment, and doing their duty, it is believed, as well as any similar body in the world, whether that body shall be replaced by a new one ?

An alternative to a body of Government engineers is the method pursued at South Pass, and proposed for Galveston; namely, for Congress to make a contract, without competition, with some individual who has influence enough to secure the passage of the necessary law.

A legislative body is notoriously a bad organization for making contracts.

But suppose the precedent followed, to be applied not to one, but to a dozen harbors. The men most plausible, most reckless in the use of statements, promises and favors, and able to secure the most powerful lobby, would be the ones who would obtain the legislation and secure the contracts. While human nature is what it is, corruption in such a scramble would be inevitable, and all who desire that public moneys shall be honestly used must dread the certain result.

A. P. Boller, M. Am. Soc. C. E.—It is difficult, if not impossible, for one inexperienced in the practical working of the jetty system, to rightly estimate the radical differences of opinion that apparently exist between Mr. Corthell and Col. Merrill, particularly as they do not agree

upon all the facts. There is a startling difference of opinion expressed by the figures made up by the army engineers for the improvement of Galveston Harbor and those proposed by Mr. Eads to Congress for doing the same work. It is inconceivable that Mr. Eads should expect to make a profit of $4 000 000 in an $8 000 000 contract. The relative plans are not sufficiently set forth to base an independent judgment upon, but it must not be forgotten that the army figures are purely an estimate, which, if wrong, only means additional appropriation, while Mr. Eads' figures are a business proposition, which must cover numerous contingencies, which, if not provided for, might result in great private disaster. Further, Mr. Eads has the prestige of a substantial success in the completion of the South Pass Jetties, undertaken against the most persistent army opposition. It would therefore be manifestly unjust to draw hasty inferences from the above figures. Col. Merrill makes a good point against Congress entering into a private contract for improving Galveston Harbor, and very justly observes that if the Government desires to go outside the army engineers, opportunity should be afforded for other engineers and contractors to tender for the work, or it should appoint Mr. Eads engineer-in-chief of the work, without the burden and responsibility of the contractor at the same time.

Incidentally, Messrs. Corthell and Merrill have brought out in their discussion a matter in which all engineers are interested, and that is the engineering conduct of the public works, a subject that has been more or less under private discussion for a long time past, and is likely in the future to be more formally considered. It is sincerely to be hoped that in this Society at least, such discussion may be conducted with perfect fairness and freedom from all personal feeling, as becomes the treatment of a delicate subject and the dignity of the profession. As to the relation between the Engineer Corps of the Army and their civil assistants, most of us have received our impressions at second-hand, and may have arrived at conclusions that this discussion will largely modify.

The Government disbursements for the civil works under its charge have grown from small beginnings to a very large annual sum, and it is but natural that the civil engineers of the country should feel that such works should be committed to their charge, and taken away from the Military Department of the Government. In these days it seems as if it ought not to be a question of argument as to the relative qualifications for such works as between the civil and the military engineer. It stands to reason that men trained and experienced in civil works are the men to whom such works should be committed, as against men trained for a military occupation, with engineering as an incidental study. It is frankly admitted in this place that the army has given the country some able engineers, while civil life has furnished many engineers by no means an ornament to the profession. But in dealing with

general principles exceptions must be ignored, and extremes should not be selected as types. There are few engineers gifted by nature with the "engineering instinct," and most of us only become qualified through the technical schools and after experience, on which basis the fitness of any engineer must be judged.

The Engineer Corps of the Army is rightly regarded as a body of picked men, having had exceptional advantages in education; but it is their misfortune, and not their fault, that the system under which they work is not so gainful of practical experience, or personal acquaintance with actual work, as is obtained by the engineer in civil life. I have yet to hear of the young Lieutenant of Engineers going out in the field as rodman, and working his way up as a subordinate through the various grades of a field party; or passing through the elementary construction work in connection with such field experience as is vitally necessary for sound constructive knowledge. The time was when West Point stood alone as a school furnishing any technical training whatever. The time is when it is the exception for a college not to have an engineering annex of some kind, to say nothing of those great schools which give to special branches of engineering as much time as West Point does to its whole miscellaneous course. It would be an admirable thing if the West Point course was supplemented by the Government detailing its picked engineer-graduates for a four or five years' post-graduate course to the various public works and surveys throughout the country, entering as rodmen, and working their way along with the rest of their civil associates—for the time being one of them, untrammeled with the restraints of the military system.

It may be remarked that the public works of the Government are open to civil engineers, large numbers of them being employed. That is true; but how are they employed, and what standing is accorded them? If any one will turn to an article printed in *Lippincott's Magazine* in August, 1883, by the late D. Y. Carpenter, he will find an unexpected revelation as to how the military system of engineering is carried on with its civil adjunct. If the arraignment of that article is true (and I have never seen any denial of it), it must be confessed that the treatment of civil engineers by the army engineers is humiliating in the extreme, against which this Society should earnestly protest. "The page slew the boar, and the knight had the gloire," seems to be the present relation between the military engineers and their pages, "the assistant engineers," a relation it will be difficult, if not impossible, to change, except by utterly disassociating the military and civil elements, as it is hardly conceivable that the Engineer Corps should open its doors to any excepting through the portals of West Point. It is therefore pertinent that the civil engineers of the country should thoroughly understand this matter, and use their best efforts toward securing a proper Government recognition of their work in some form

that would be consistent with their self-respect, and place them on such a footing that it is not a question of rivalry or comparison between the military and civil engineer, but one of equality. They should be two arms of the same Government, one the arm of war, the other the arm of peace; the one concerned with fortification and defense, the other with the improvements of harbors and rivers, the care of light-houses and conduct of surveys. The Board of Military Engineers and the Board of Public Works—a Board of Public Works conducted on the strictest civil service principles, open alike to army, navy or civil engineers.

Capt. J. C. Post, M. Am. Soc. C. E.—I have read the valuable paper by Mr. E. L. Corthell on "The South Pass Jetties," and its interesting and careful review by Colonel Merrill. Both of these papers are important contributions, and discuss one of the most difficult problems of hydraulic engineering.

Mr. Corthell, reasoning from his experience in guiding the waters of the Mississippi between jetties, draws his conclusions—some eight in number—as to the manner in which bars on sandy coasts should be treated. Colonel Merrill takes issue with him in regard to some of these, especially where he refers to the application of jetties now in progress at Charleston and Galveston. In his paper Mr. Corthell seems to have overlooked the great difference between the problem to be solved at the mouth of the Mississippi and those at the points named. On the one hand a mighty river furnishes the scouring currents, while on the other the ocean supplies them. The water from the latter must first find its way into the harbor before it can be gathered together and used for creating a channel.

Having had an experience of over five years upon the works at Charleston, and similar ones at Cumberland Sound and the mouth of the St. John's River, it has occurred to me that a brief statement of the manner in which bars at the entrance of harbors on low sandy coasts should be improved, as a result of my studies, would be of interest to the Members of the Society.

In the improvement of harbors of this character, there are but three methods which can be used by the engineer. First, dredging; second, the construction of works to concentrate the flow of the tide upon the bar, and thus create a channel; and third, the construction of a canal with locks, connecting the harbor with the sea.

The first is usually but a temporary expedient, and seldom gives much, if any, permanent relief. The third is objectionable as causing constant delays, and being limited in capacity. The engineer is therefore reduced to the second method, which is the only one that can satisfy all the conditions required. This, however, is by no means certain of success, and can only be properly applied after a thorough and complete understanding of the locality. It should be borne in mind

that if it fails, it will be a most costly experiment. In that event, the only means left for improving the harbor is by the third method, and this must be accepted with all its limitations.

In preparing a plan for the improvement of a bar by means of concentrating works, or jetties as they are commonly called, the engineer should first study the action of the tidal currents, their strength and direction; the location of the existing channels, and the position of the deep water inside the bar; the height of the bar, the materials of which it is composed, and the slope on its exterior or sea face. He also notes the direction of the prevailing storms. It is upon these latter, and the manner in which the tidal wave approaches the coast, that he must rely for littoral or dispersing currents.

After considering these various conditions, and the width and depth of the channel necessary for the accommodation of commerce, all other things being equal, he locates his works so that they may obtain the required results with the least strain upon themselves. In endeavoring to satisfy this condition, he can only determine their position approximately, and it is left for him to ascertain, as the work proceeds, the height to which they should be carried to secure success. He is fully aware that the lower the level at which they can be kept, the more economically they will be completed; and at the same time they will be rendered less liable to damage during violent gales. A broad foundation is essential, whether the jetties be high or low; and this, in construction, must be kept considerably in advance of the raised portion, unless scour is desired for some particular reason, such as to reduce a shoal in the path of the work, upon which the seas break so heavily that the extension of the jetty is impracticable. Scour under these circumstances has also the advantage of permitting the foundation to be placed at a sufficient depth to be protected partially, if not wholly, from the action of the storm waves. As a general proposition, however, scour in advance of the work should be avoided as much as possible, as it adds to the ultimate cost.

I believe all jetties for the improvement of sandy bars should be raised above the level of low water. In localities where the range of tide is small, and the tidal basin of limited dimensions, and it is desired to form a channel of large proportions, it will be necessary to concentrate all the current action by bringing the jetties to the level of high water. It may even be required, in order to preserve the channel, to raise the jetties above this level, and obtain the aid of the abnormal currents produced between them by the storm tides, and thus drive back and disperse the drift bar that may be forming at their entrance by the action of the heavy seas.

At other places, where the tidal range is considerable, and the basin to be filled is large, it is probable that a channel of sufficient size may be produced without raising the jetties to the level of high water. If

they are carried to a height above that necessary to maintain the channel, they will form a greater obstruction to the inflowing tide, and the basin cannot be completely filled during the tidal interval, unless the dimensions of the channel be increased beyond those needed. This would produce an unnecessary scour by the flood tides, and consequently a greater deposit in the harbor. It is probable that with all jetties, whatever their height or distance apart, the flood tide will cause some shoaling in the harbor. This may or may not be injurious. The amount of sand, however, to be moved by this tide should be kept at a minimum, to avoid, as far as possible, deleterious effects.

Jetties that are raised too high also cause the ebb currents to remove a greater quantity of sand than is necessary, thereby increasing the probability of a bar forming in front of them.

To give an idea of the force which the engineer has at his disposal at some of the places where improvements of this kind have been undertaken, the area of the tidal basin and average range of the tide at each have been tabulated as follows :

Locality.	Area of Tidal Basin.	Average Rise and Fall of Tide.	Tides in 24 hours.
	Square Miles.	Feet.	
Charleston	15	5.1	2
Cumberland Sound	25	5.9	2
St. John's	16	4.3	2
Galveston	451	1.2	1

By multiplying the area, reduced to square feet, by the range of the tide, a fair approximation of the amount of water flowing over these bars during an average tide may be obtained. During the ebb tides this will be reinforced by the fresh-water discharge of the streams emptying into the basin, and the strength of the currents will be correspondingly increased. It is usual for the ebb currents to commence immediately after what is known as high-water slack, and attain their greatest velocity during the second and third quarters of the tide. An important variation from this rule has been observed at the mouth of the St. John's River, where the ebb currents do not appear until about two hours after the water has begun to fall, or until it has fallen one-third of its range. This fact will enable the jetties at this place to be kept below the level of high water without losing the effect of the ebb-tide currents.

I will here remark that the St. John's River is a sluggish stream, with a comparatively small fresh-water discharge. The bar at its mouth has therefore been classed in this discussion with the bars at Cumberland Sound and at Charleston Harbor.

In conclusion "three," Mr. Corthell states "that it is necessary to raise jetties high enough above the surface of the water to prevent the sands from washing over the works by the force of the storm waves." Undoubtedly this may be true for some localities, especially where the channel between the jetties is narrow, and there is a large amount of drifting materials, but when the distance between them is nearly half a mile, or more, and the scouring current strong, it is highly improbable that the channel will be appreciably affected. Jetties of a sufficient height to produce and maintain a channel of such a size, will probably give all the protection against this action that is necessary.

In opposition to the general theory stated in conclusion "three," I will call attention to the channel that has been formed at Aransas Pass by a single jetty. This channel is not protected by the work, but is on the side from which the drift comes, yet it is maintained by the currents conserved and guided by the jetty. Again, at St. John's Bar, there is a good channel on the north side of the jetty now being constructed, with no works to protect it from the prevailing northeast storms. Since the work began, this channel has materially increased in depth, due to its influence.

The principal difficulty encountered in retaining at a low level, jetties built upon sand, results from the overfall which acts to undermine them. This may require that they should be given additional height. The amount of this increase will depend upon the action of the tidal currents at the locality.

Low jetties have been extensively used in improving the harbors along the continental shore of the North Sea, but only with moderate success. This coast is low, and full of shoals, and the movement of the sands by the action of the tides and storms very great. These harbors are without natural tidal basins, those existing originally having been reclaimed.

The jetties generally gave an increase of depth, but owing to the littoral drift the shore line advanced with the work, and the benefit was small. Some of these jetties were extended in hopes of increasing their effects, but the shore line again advanced to their ends, and little or nothing was gained. Artificial sluicing basins were then constructed, and finally dredging has been resorted to for increased depth. It is claimed by those engineers who have made a special study of these harbors, that high jetties would have caused a more rapid accumulation of sand. The rise and fall of the tide along this coast varies from 20 feet 6 inches at Calais, to 14 feet 4 inches at Blankenburghe.

One of the jetties at the mouth of the Liffey entrance to Dublin Harbor has also been kept at a low level along its outer portion. This river is a comparatively insignificant stream. In describing this work, Mr. Griffith, in a paper read before "The Institution of Civil Engineers" in 1879, gives the following reasons for retaining part of the jetty at a

low level: "Had the great north wall been raised above high-water level for its entire length, there would have been a strong current through the entrance during the whole of the ebb. In that case the current on the first half of the ebb would become a sand-bearing stream, and on meeting the opposing bay current would either deposit sand and form another shoal, more difficult to deal with than the bar, or be deflected to the southward, and re-enter the harbor on the flood, bringing back whatever sand was still held in suspension."

The original depth of the water over this bar was 6 feet 6 inches at low tide, and it has been increased to 16 feet, with a probability of obtaining 18 feet. The area of the tidal basin is about four square miles, of which only one-quarter is included within the line of low water. Reclamation has been avoided within this basin, and dredging has been continued to remove a shoal that has been forming since the construction of the north wall. By this means the tidal capacity of the harbor has also been improved. The rise and fall of the tide in this harbor is 13 feet.

By a comparison of the two localities cited, the south shore of the North Sea and the coast of Ireland, where low jetties have been applied with such different results, it will be observed that the absence of tidal basins in the continental harbors seems to be responsible for the limited success at those places. An endeavor has been made to supply this deficiency by constructing sluicing basins. These, however, can only be made of comparatively small dimensions, and to obtain their greatest effect the water was allowed to flow from them when the tide was lowest. It is not improbable that the violent currents that were produced in this manner removed more sand in the vicinity of the basin than it could convey through the jetties, and thus shoals were formed between them. Even if these currents were successful in removing this sand beyond the works, it was carried into the sea at a time when the littoral currents were at a minimum, and much of it, no doubt, was deposited at the entrance of the jetties to form a bar, or to be carried back again into the harbor by the next flood tide.

In the sixth conclusion I agree most heartily with Mr. Corthell, for in no class of construction is rapid and energetic work more essential than in the improvement of a sand bar. Unfortunately the present system of appropriating for these works will not permit that, and there are long intervals during their construction when nothing can be done for want of funds. Take for example Charleston Harbor, where the original estimate, based upon there being continuous work, was $1 800 000. This improvement was begun in 1878, and up to date, a period of seven years, the total amount appropriated has been $1 290 000, which has allowed an average yearly expenditure of $184 000. For the improvement of Cumberland Sound, the estimate under the same conditions was $2 071 023, and during the five years this work has

been under way, there have been appropriated but $255 000, or sufficient for a yearly expenditure of $51 000. Again, at St. John's Bar, where the estimate was $1 306 409 for rapid and continuous work, which was well known to be specially essential at this place, because of the sudden and constant changes in the bar, during the five years since the commencement of these jetties, but $525 000 has been appropriated, and an average yearly expenditure of $105 000 has been made. By experience, it has been ascertained that $50 000 may be expended during a fair average month's work at these places. Comparing this with the amounts that have been available, it can readily be seen how much time might have been saved if these works could have been carried on continuously in the proper manner. Thus, at Charleston, what has been done in seven years should have been done in two years; at Cumberland Sound, what has taken five years should have been done in five months; and what has been done in five years at St. John's Bar should have been done in ten and one-half months.

Any one conversant with this class of works, is aware that the greater portion of their cost is in the construction of the foundation, and raising their levels to where the effect begins to be felt. This may require an expenditure of 75 per cent. of the estimated cost, or perhaps more, depending upon the locality; and this is true where the works are pressed forward in the most vigorous manner. But where they are delayed, because of insufficient means, as in the cases alluded to, and operations are suspended from time to time, and the work necessarily abandoned, while the currents are continually at work, causing changes that are detrimental to the portion already constructed, creating holes to be filled up, and otherwise adding to the cost of completing the jetties, a much larger percentage must be consumed; and even the whole of the original estimate may be expended before the sands begin to move. It is by these delays that works which should be completed in two or three years, or at least give some beneficial results in that time, are made to drag along from year to year. The engineer, in the meantime, is compelled to report "no results," well knowing that he is being judged adversely by an impatient public, who hold him alone responsible. It has no conception of the magnitude of the undertakings, nor of the time necessary for their completion, and cannot be made to realize that these enforced delays are increasing the ultimate cost of the work, and the time of completion, by amounts which cannot be estimated. The engineer's plan may be the best that can be adopted, one well qualified to give success, and yet, through no fault of his, he is charged with failure, because he does not fulfill the anticipations of those who do not fully comprehend the difficulties with which he contends.

I have now stated the conditions under which the river and harbor works in general have been conducted throughout the country, and which have added to the cost of all, and particularly to those where

improvements of sandy bars by jetties have been undertaken. I trust that I have made myself sufficiently clear to cause every one to appreciate the unusual embarrassments that beset these works under the present system of making appropriations. Engineering difficulties it is the duty of the profession to meet and overcome, if practicable, but it is the financial part—the insufficient and irregular amounts, that are given without due regard to the importance, the character, or economical construction of the works—which particularly worries and harasses the engineer, who feels that his reputation is at stake, and that he may be seriously injured professionally if his works fail from any cause.

Considering all that the Army Engineers have had to encounter in conducting these improvements, I cannot see how any class of men could have done better, and I believe that the Members of this Society, when they fully understand the situation, will even be led to express surprise that so much has been accomplished under a system so faulty and so full of perplexities.

With regular and ample appropriations, a large portion of the works would by this time have been completed; but instead of making a provision to this end, Congress has seen fit again this year to withhold appropriations for the river and harbor works, and many of them, especially those most important, will be obliged to pass through another period of delay, peril, and deterioration.

Edward P. North, M. Am. Soc. C. E.—In the paper under consideration, Mr. Corthell says: "A lesson may be drawn from this matter concerning the proper mode of carrying on public engineering work. That lesson is, as it appears to the writer, that the Government should summon to its aid the best engineering talent within its reach, and should not give the general or exclusive charge of public improvements into the hands of engineers educated to conduct works of a totally different character. Let the doors be thrown open freely to all engineers fitted by education and experience for the service required."

It may be well to submit, in this connection, that the whole system under which our river and harbor improvements are prosecuted, could be advantageously remodeled.

As long ago as 1808, the then Secretary of the Treasury, in his report to the Senate on "Public Roads and Canals," after estimating $16 600 000 for works of public utility, adds: "As the expense must be defrayed from the general funds of the Union, justice, and perhaps policy no less than justice, seems to require that a number of local improvements sufficient to equalize the advantages should also be undertaken in those States, parts of States, or districts which are less immediately interested in those inland communications. * * * * Without pretending to suggest what would be the additional sum necessary for that object, it will, for the sake of round numbers, be

estimated at $3 400 000
Which added to the sum estimated for general improvements..... 16 600 000

Would make an aggregate of..... $20 000 000"

Thus proposing to add over one-fifth of what, in the slang of the day, is called "grease," to the amount necessary to the appropriation to accomplish the end sought.

The unfortunate precedent thus established, has seemingly carried force through all future considerations of a subject so vital to our prosperity, and States and districts on principles of policy, no less than justice, still require local improvements, some of which are not calculated to add to the wealth of our country.

In the meantime, such a clamor has been raised against appropriations for the improvement of our rivers and harbors, particularly those that have a tendency to cheapen distribution among ourselves, and control freight rates on railroads, that unthinking persons hold any attempt to increase the wealth of our country by improving the avenues for water transportation as disreputable and a *prima facie* evidence of fraud, and our legislators, grown careless of censure so liberally bestowed on, in general, beneficent enterprises, and alighting alike on the just and the unjust, in some instances, at least, following Gallatin's precedent to its logical conclusion, have, in the spirit of policy, no less than the spirit of justice, endeavored to add to their popularity and political strength by securing large appropriations regardless of their utility, or even withholding them entirely from unappreciative sections, as a mode of punishment.

The ordinary course of river and harbor improvements is about as follows, viz.: An appropriation is secured, either with or without previous survey, and a plan adopted having in view the supposed requirements of a locality, and also the fact that it is uncertain whether the next year will give an appropriation or not. Under these circumstances the money, even when in the hands of a competent engineer, is extremely liable to be expended tentatively, and without proper consideration of the necessities of the case, and to poor advantage; so that we read in the Report of the Chief of United States Engineers for 1883, of the improvement of the Fox and Wisconsin Rivers, for which, since 1870, $2 483 723.97 has been appropriated: "The whole subject of the improvement of the Fox and Wisconsin Rivers has been referred to the Board of Engineers for Fortifications and River and Harbor Improvements, and it is expected that the Board will make a report in time for the next Session of Congress.

"Future work on the improvement will depend in a great measure on the report of the Board."

However unsuited for the locality the plan adopted may be, all ad-

verse criticism is apt to be vigorously reprehended, through fear that local interests may be endangered by prejudicial reports from the engineer-officer in charge, it being only after years of disappointment that the community rises to the sublime height of refusing an appropriation, or at least requesting that one be not made.

If, under the pressure of any dissatisfaction, a commission is appointed, it often has for its head the very man who is responsible for all the bad engineering of the district, whose record must be kept in view and protected, while an *esprit de corps*, as in the case of the deliberations over the relative merits of a canal and jetties at the mouth of the Mississippi, is apt to seriously warp the judgment of the Board.

If, for the present arrangement, a mixed commission, consisting of, say, three civil and three military engineers, with three or four persons interested in the commerce and industries of the locality, was uniformly appointed, who should be instructed to report on the necessity for the proposed improvement, on the plan to be adopted, and on its costs, Congress could intelligently consider the advantages of the proposed improvement, and at one time appropriate money enough to complete the work with the speed necessary to economy, without yearly recourse to those sectional considerations introduced by Secretary Gallatin's report.

The elaborate definition of civil engineering as adopted by our brethren of England: "The art of directing the great sources of power in nature for the use and convenience of man, as the means of production and of traffic in States, both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange; and in the construction of ports, harbors, moles, breakwaters, light-houses, and in the art of navigation by artificial power for the purposes of commerce; and in the construction and adaptation of machinery, and in the drainage of cities and towns," will be quoted oftener than Ashbel Welch's "that engineering is best which most fully answers its purpose at the least cost;" or our Past-President Whittemore's definition: "Engineering is economy in time, space and money."

But successful engineering work is done on the American definition, even by those who have never heard it, and probably the greatest advantage in the plan of a mixed commission would lie in the Government securing the services of men used to considering the economic connection between expenditures and the returns probably derived therefrom; men who have also shown their fitness for the profession by the position they have attained rather than by marks obtained in any school, and the sure results of promotion by seniority.

The justice and policy of having the business interests of the locality represented, seem too patent to require further discussion.

It is not held that the proposed commission would be infallible, or that no mistakes would be made by it; but under it, it is doubtful if over $3 500 000 (including simple interest at 5 per cent.) would be again expended through 15 years, without even completing a local improvement, before the receipt of a report on which the future work on the improvement depends.

Nor would the practice of building parallel piers for our lake harbors be continued, affording a clean race for waves past the business part of the town, and entirely ignoring the advantages of a fore-shore.

Neither would we see the entrance to another harbor, intended to accommodate center-board vessels, 38 to 40 feet wide, with a fixed keel only 1 foot deep, contracted to a width of 125 feet, while the shore sides of harbor piers, on the same coast, opening 250 feet, were ornamented with wrecks of schooners, that, on account of the necessity of drawing up their center-boards, had failed to make an entrance.

And no other harbor would probably be commenced in hard pan where the first year's dredging cost $1.20 per cubic yard, while 100 yards to the north was a swamp where the dredging subsequently cost 12 to 13 cents.

If Congress could be induced to appropriate at one time the funds necessary for the completion of the work, limiting the yearly expenditures as might be thought best, cases like that at Ontanagon would be less frequent, where dilatory piers have followed a slowly advancing bar since 1867, without material amelioration of its environments, with, however, an expenditure of $257 600, and this under frequent calls from the engineer in charge for larger appropriations.

The malign influences of small and ineffective appropriations, consequent on the clamors of local jealousies, or supposed political necessities, cannot be measured alone by the greatly-increased cost of the work directly affected; but by affording a ready and readily accepted excuse for expensive management, they have a tendency to eliminate that pride in economical management that is the basis of successful engineering.

The extent to which the charge of public works should be thrown open to civil engineers is a question that will probably have many different answers, depending on the mental habit of the answerer, his knowledge of the subject, and his leaning to either the English or the American definition of civil engineering.

The engineer-officers of our Government have a long and honorable record for financial probity. Some members of the corps may have been lazy, inebriate or impracticable, but it is not charged that they have been influenced by dishonest means, except, it is believed, in one instance. The very fact urged against them, that they have only to live to attain the summit of their professional possibilities, and while they live their support is assured to them, not only removing temptation

from them, but allowing them to follow their convictions, right or wrong, with a supreme indifference to the consequences, is not entirely devoid of professional and economical advantages. The contrast between the City of Washington when its public works were under the charge of a political commission, and the same city under an engineer commission that the politicians cannot touch, or, touching, cannot hurt, is an illustration patent to our whole profession.

It seems that a modification of the law making the senior engineer-officer in charge of a work or district responsible for the funds appropriated, might be made with great advantage to the service, by the appointment of paymasters, only requiring the engineer to certify bills and designate the accounts to which they should be charged, thus giving the man responsible for the conduct of the work and the economical results obtained, an opportunity to know of its details.

The unfortunate prostitution of the talents and time of the senior officer to the office and affairs of a book-keeper, was as long ago as 1868 referred to as follows: "If it were possible to relieve the superintending engineer, who disburses the money belonging to as many different appropriations as has been my lot, from a portion of the duties now intrusted to him, it would enable him to devote a part of his time, now devoted to paper work, to other and more important professional duties."

This protest against clerkly labor seems to have been unavailing, and the superintending engineer, if he is wise, first sees to his accounts, and then sees to the work for the superintendence of which he is supposed to have been educated.

The ill effects of the lack of practical knowledge of work born of this state of affairs do not noticeably come to the surface when dealing with the large resources of our Government, but when handling enterprises organized on the basis common to private or corporate affairs are sometimes disastrous. As when one of them being appointed Chief Engineer of an intended great trunk line from the Mississippi Valley to the Pacific Ocean, falling into the hands of a crank, smashed his company by recommending a narrow-gauge railroad; or another, who it is said, after the expenditure of $1 000 000 on a piece of property, left it less ready for use than when he took charge of it.

It is possible also that an improvement might be made in the education of these engineers. For, according to a writer in a late number of the *Engineering and Mining Journal*, after leaving West Point they are again confined in an additional school for two and a half years, where associating entirely with men of one caste, it is nearly impossible their minds should not become more or less contracted.

If these young men on leaving their schools could be placed, as the graduates of other technical schools are, in direct contact with their work, they would doubtless be much better acquainted with the prop-

erties of time, space and money than on their final discharge from semi-monastic surroundings.

It does not seem to the writer that the practice of Government summoning to its aid the best engineering talent within its reach, could, on the whole, work adversely to its interests.

There would, doubtless, be a certain loss of *esprit de corps*, but that, so far as it leads to the support of unsuitable plans and inefficient management, would not be an unmixed evil.

There could hardly fail to be a sure gain from the introduction of men trained in different schools, even if it were admitted that their training was not better and broader than that given by our own Government. But if the service should carry a rivalry between those fitted under different systems, that rivalry could hardly fail to be one for excellence of plans and execution.

Comparing the results at the commencement of our late unpleasantness, when the regular officers were supposed to possess all warlike knowledge, with those attained at a later period, when volunteer officers held important commands, few can doubt the mutual gain to both classes, as well as to the whole country, from the employment of volunteers. And it seems probable that still more advantageous results would accrue from throwing the doors open to all engineers fitted for the service required.

Robert E. McMath, M. Am. Soc. C. E.—Concerning the South Pass Jetties themselves I have nothing to say, but I do wish to say something as to the application of the supposed lessons of the jetties to the Mississippi at points where the conditions are essentially dissimilar.

The jetty experience is good, so far as it goes, and it goes far enough at the Passes to satisfy all reasonable demands; but it comes very far short of furnishing a precedent for practice in dealing with the river at the inland parts of its course. At the jetties but a small part of the volume of the Mississippi was dealt with; the range of stage is but small; and the ratio of volume change in the South Pass is a low one. The concentration there by the jetty works in space and width, was followed as a result by a gradual and abiding increase of volume, much as if drawn from a reservoir through an enlarging orifice; so that the channel, while forming and after, was always full to near the same level, and the current did not vary in velocity to the extent which currents do in the river higher up, and but very little in direction. The normal volume, in the true sense of the term, was there an increasing quantity. There is no warrant to predict from such an experience that useful results would follow from the action of floods under concentration in time as well as space; for, as the result of such concentration, the true normal volume must be diminished.

The application of jetty lessons to levees, as if these were similar,

though dealing only with a passing flood, to the jetties, which are always exerting their full influence, is an error. The principle I have sufficiently discussed in "Levee Theory Tested by Facts."*

Another attempt to transfer jetty experience to works in the channel of the Mississippi without allowing for the difference in essential conditions, resulting in practical failure, may be found in the training dikes or jetties, by which a reduction of the wide places to an approximately uniform width has been attempted. Though the principle, as laid down on paper, was that the Mississippi was to be subjected to a high-water treatment, the dikes were located in the bed, and having their bases in water at low stages or on low bars, the limit of possible construction left them submerged at high stages and so under the condemnation of Mr. Corthell's third lesson. No "high-water treatment" doctor has ever shown how that treatment could be applied to the Mississippi practically. The so-called jetty principles are not applicable to the Mississippi in general; for, if placed on the bank as levees, they affect only abnormal and irregular conditions which are not reconcilable with the formation and maintenance of a water-way adapted to a smaller volume, which must be admitted to be the normal one, and with the normal water-way, improvement of navigation has solely to do; and if applied in the bed as training dikes, the "high-water treatment" "unsubmersible" idea would require that the dikes be parallel permeable walls fifty or more feet in height, works which would not commend themselves to engineers as stable structures, if their construction were possible.

Since the applicability of the jetty system to river improvement has been broadly claimed, and has been too readily accepted, by engineers as well as by the public, as a delightfully simple and eminently practicable method, it may be well to make the essential difference of conditions plain.

At the Passes the effort to control the river, compared as to difficulty with that at the middle portions of its course, is much like a trial of strength with an adversary in his old age as compared with the vigor of his manhood; or, if the thought of using the river's own forces is suggested, the river at the Passes is as an ox, in the upper portions it is as a wild bull; for at the Passes the destructive power of the Mississippi is almost exhausted, indeed little erosive work is done for many miles above the Passes.

At the junction of the Ohio with the Mississippi, low-water elevation is about 290 feet above gulf level; at high water the elevation is 340 feet, and the volume of water twenty times greater than at low. The mass of water, multiplied by its elevation above sea level, represents a definite capacity for work, potential energy; when the water reaches sea level its capacity

* Levee Theory Tested by Facts, by Robert E. McMath, M. Am. Soc. C. E., Transactions Am. Soc. C. E., No. 291, Vol. XIII, October, 1884.

for work is zero, that is, the whole has been expended in some kind of work somewhere. The resistances of a channel 1 060 miles in length, due to wetted surfaces and bends, together with the burden of transporting the solid matter suspended in the water at the Ohio junction, measure the necessary work. The difference between capacity and necessary work represents surplus energy, which must be expended in internal work, whirls and commotions, and external work on bed and banks and in the transportation of eroded material. Viewed in this hydro-dynamic light, the instability of river banks and shifting channels, which characterize and constitute silt-bearing rivers, are the consequences of surplus energy; that is to say, the fall in the river is greater than is compatible with the conveyance of the constantly varying volume of water in a fixed channel traversing alluvial material. Moreover, since erosion practically ceases below Red River, this surplus fall is located and the surplus energy is nearly all expended in the 765 miles between the Ohio and Red Rivers. Reduce the river to uniform width, or whatever other ideal condition you please, and the dynamic problem, How shall the surplus energy be expended? will remain, and most of the ideal improvements proposed for the river will be found on examination to increase the surplus.

At the Passes the potential energy of the mass is small, and there remains little or no surplus for destructive work; elsewhere the surplus is very great. The existence or non-existence of surplus is the essential difference of condition to which I would call attention, for it explains, not only why the jetty plan of concentration has failed, and must fail, to accomplish a permanent improvement of the Mississippi, but it also disposes of all the rival plans and suggestions that have thus far been brought forward.

The principles involved in using the dynamic condition as a criterion to judge plans by are simple, and need only to be named.

The fact that erosion goes on actively at the lowest stages, as far down as the mouth of the Red River, proves that there is a surplus of energy even then. When both mass and elevation are increased by floods, this surplus is increased manifold. A further increase must result from concentration of flood in time and space by levees, therefore confinement of flood waters by levees must serve to further increase an energy already much too great. Outlets for part of the water, so far as they even tend to lower the surface at the point of depletion, increase the slope and surplus energy in the part of the river immediately above the outlet, and, as a next consequence, bring an increased quantity of scoured material into the part next below the outlet, where it causes fresh evils. Cut-offs, by shortening the course, increase the ratio of fall to length, and so intensify the action of surplus energy.

Thus levees, outlets and cut-offs, confining, depleting and straightening, though differing widely in object and instrument, all have the common feature that they increase the residue of destructive energy,

and hence are all incompatible with the stable condition which must be attained by a permanent improvement.

Now, if this surplus energy was applied to deepening the bed, the river would be lowered, the fall in the lower end reduced, and destructive action would there cease. But the fall lost below must be concentrated in the upper reaches with increase of destructive power, or at a cataract where energy would be expended in internal work. Some seem to think such a result desirable and probable. Once there was talk about "setting Cairo on a hill," by extending the conditions below the Red River up-stream. Geologists say that an arm of the gulf once occupied the entire area now covered by alluvial deposits, and whose waves washed the hills above Cairo. The theory of those who would extend deep water up-stream implies an expectation that, by the magic of a few willow wands, the course of nature for geologic ages will be reversed, and the river will be seen lowering itself in a bed which it has been raising and extending since its history began.

I have shown by facts in "Levee Theory Tested by Facts," pages 347 and 348, that low-stage action lowers the river in its bed locally for the time, but that high-stage action raises it, which is proof that surplus energy works evil. The high-stage action predominates, for the Mississippi River Commission Report for 1883, pages 281–286, furnishes facts showing increase of width. This is also proof that surplus energy in time of flood is expended in lateral erosion rather than in vertical scour. As the river widens it must shoal, else sectional area and channel capacity would increase, and from such increase less necessary work and more surplus would surely follow, a tendency which conservative adjustments will not allow.

The dynamic view, hinted at rather than presented in this discussion, is not sufficiently developed to warrant the present bringing forward of a plan to replace those it condemns, though the general character of such a plan is plainly indicated. It must be based on equalization of volume, as that idea stands in contrast to the concentration of volume, which the jetty lessons have been supposed to favor.

Mr. Corthell's ninth and final lesson touches upon a large subject, and one not peculiar to jetty experience.

A. C. Savage, M. Am. Soc. C. E.—Can the same conditions be assumed to exist at points along the channels of the great alluvial streams that render the jetty system the most proper for the mouths of these streams?

Without having the figures at hand for present reference, I would say that the increase of the flood volume of the Red River due to the removal of the raft above Shreveport, La., and the partial closing of the auxiliary channels on the west side of the stream, have resulted in the scouring away of a much wider channel than formerly existed, and in the

increased overflow of the adjacent country to such an extent as to cause the total abandonment of a number of plantations which were formerly in a high state of cultivation.

This state of affairs exists to such an extent as to cause the examination during the past year of Bayou Pierre, to ascertain the feasibility of reopening that channel for navigation and so relieving Red River.

Without pretending to decide so important a problem, it would appear to me that a system of auxiliary high-water channels, in conjunction with the judicious straightening of the main channel, would tend to a solution of the difficulties presented by those streams.

Paper 291, Trans. Am. Soc. C. E. "Levee Theory Tested by Facts," by Robert E. McMath, M. Am. Soc. C. E., seems to me to present conclusive arguments against an exclusive system of concentration by levees.

That dikes or levees are available at many points to correct local troubles, is, I think, undoubtedly true.

That a complete system of levees for the Mississippi, or for any streams of the same character, having in view the defining of a permanent channel and banks throughout its length can be made practically successful, I should find it difficult to believe.

O. Chanute, M. Am. Soc. C. E.—I think it not improbable that a good many Members have failed to realize the practical application to be made of some of the statements in Mr. Corthell's paper, and in Col. Merrill's discussion of it, for want of a little preliminary information.

This probably arises from the fact that the discussion thus far has bordered on an important public question, and that while these gentlemen have been stating general principles, they have had chiefly in mind their application to one particular place.

I take it that the real question at issue is whether the jetties as planned and partly built at Galveston, will or will not prove a success, and whether it is advisable for the Government to enter into a contract with Mr. Eads (or some one else) for the improvement of the entrance to Galveston Harbor.

Having visited Galveston this winter, in the interest of some contractors, to see if there was any money to be made by putting in an adverse proposition, and having decided that there was not, I am perhaps in a position to give such explanatory statements that the subject may be further discussed by the Members of this Society. The facts, as I gathered them, seem to be as follows:

The works for securing an entrance across Galveston Bar (over which there was then 11 or 12 feet of water) were planned in 1874 by the Government engineers. These works consisted of two jetties about 2¼ miles apart, the South Jetty being planned abreast of the northerly end of Galveston Island (on which the city is built) and

the North Jetty being located some distance above the southerly end of Bolivar Peninsula. The distance between these two points of land (Galveston Island and Bolivar Peninsula) is about 9 000 feet, and in this "gorge" the sweep of the tides, going in and out of Galveston Bay, which covers some 451 square miles of area, maintains a channel of at least 30 feet of water, there being some holes as much as 50 feet deep.

The crest of the bar is about 4 miles out from the shore, and with a few variations of level, it is continuous, thus cutting off access to the harbor, except for vessels of shallow draft, and compelling much of the commerce of the port to be lightered out to ships lying in the offing, at a cost of some $2 a ton.

The works as planned, were expected to produce a depth of 18 feet across this bar, and this was then thought sufficient for the commerce of the port. Since then, the benefits which have resulted by the Jetties at the South Pass, in giving 30 feet depth to New Orleans, have made it evident that it is advisable to secure more than 18 feet of water into Galveston Harbor, if possible.

Work was first started upon the North Jetty, and a portion of it was built. This was attempted to be done with "gabions," these being great baskets of woven brush, some 9 feet long, and plastered over with cement, with plank bottom and top, and filled with sand, after they were placed in position.

The experiment failed; the gabions were buffeted by the waves, destroyed, carried away, or swallowed up by the sand on which they rested, and all trace of them has now disappeared. The plan was given up in 1879, after spending a large sum of money, and nothing now exists of this North Jetty.

In 1880, plans were made for building these jetties of brush mattresses, weighted and anchored down with rip-rap stone, at an estimated cost of $1 825 813, and work was begun upon the South Jetty under yearly appropriations from Congress. This South Jetty has now been built out 4¼ miles, upon a location differing somewhat from that originally proposed. It joins the shore at the northern extremity of Galveston Island, which is called Fort Point, and was built up to low water, except a low place about 3 miles from the shore, which was not completed when the funds ran out. This has been accomplished at a cost of some $975 000, of which the City of Galveston has furnished $100 000, under circumstances which will be stated hereafter.

This South Jetty is from 60 to 120 feet wide across the base, according to the depth of water, and was originally from 5 to 12 feet high. It is composed of alternate courses of brush mattresses—about 1.7 feet high, and generally 30 x 35, 30 x 45, and 30 x 90—so arranged as to break joints and secure a bond. These mattresses are weighted down with stone, in the proportion of about one cubic yard of stone to every three

cubic yards of mattress, and the aggregate cost has been about $3 per cubic yard of jetty. Since it was built it has settled down from 2 to 5 feet, partly by compression of the brush and partly in consequence of the ravages of the *teredo navalis*, which is exceedingly active in those Southern waters, and which eats up even the smallest twigs, if it can get access to them.

It was originally expected that the stone on top of the mattresses, the sand brought in by the waves, and the marine shells of various molluscs, would prove an effective protection. These expectations have not been realized, and a portion at least of the jetty has been eaten up and injured by the *teredo*. How far this injury has gone cannot be told without a special examination with divers, or other appropriate method, for in consequence of its settling, the jetty is now throughout from 1 to 5 feet under water at low tide, and there is 8 feet of water over the low place near the outer end.

The jetties were originally planned to be wholly submerged. I believe it was intended that there should be 3 or 4 feet of water over them at low tide. They were therefore designed to stop short of the shore where they reached that depth of water, and hence arises the idea of the "lateral outlets" near the land which Mr. Corthell criticizes, but which Col. Merrill denies. The South Jetty, the only one built, extends as has been said clear to the shore, and I believe it is now conceded that it should be raised up to low water at least, in order to be thoroughly effective.

It will be noted from the foregoing statements that the works at Galveston are in a very incomplete condition. Where two jetties were planned in 1874, but one has been built. This even is not complete, and it has been constructed neither on the location nor upon the plan originally proposed, while it extends to the shore, instead of stopping short of it.

The effect of this one jetty has been, by acting as a training wall, and deflecting the currents near its outer end, to produce a small local channel about two feet deeper than the rest of the bar, giving entrance to the harbor to vessels drawing 12 to 14 feet of water. Even this small benefit has been denied in the recent controversy, but I think myself, from local information, that it may fairly be said to exist, and to have been produced by the jetty in the condition in which it was in 1882.

Unfortunately, as it afterwards turned out, the Government engineer in charge of these works, encouraged by this partial success, and by his greater success at Aransas Pass Bar, where a single jetty, acting as a training wall, has caused a scour across the bar 6½ feet deep, was led to expect still greater results from the completion of the one jetty then in course of construction at Galveston, and when Congress failed to pass the harbor appropriation for 1883, he gave the Galveston people to understand that the expenditure of $100 000 more would effect the entire

completion of the South Jetty, and that, in his opinion, this would result in a channel 18 feet deep by the ensuing fall.

Being vitally interested in the success of these works, which had been dragging since 1874, the people of Galveston raised the $100 000 by an issue of local bonds, and turned it over to the Government, with no other condition than that it should be expended on the jetty.

The money was received and spent, but the result was practically nothing.

The people of Galveston then lost faith in the success of the Government works, and in November, 1883, a committee of Aldermen and citizens addressed a letter to Mr. Eads, then in England, asking him whether he would undertake the work of deepening the water on Galveston Bar; how long it would require to complete the works; what would be their probable cost; and for what sum per annum he would guarantee to maintain such depth as he expected to obtain?

That master of hydraulic engineering, who had up to that time refused repeatedly to express any opinion upon the probable success of the Government works at Galveston, answered to this address of the people that he was thoroughly familiar with their harbor; that a less channel depth than 30 feet should not be contemplated by the city; that he would not be willing to undertake to secure this unless entirely untrameled in planning the works, and all needed appropriations must be secured in advance; that he should make a very different location of the works from that of the Government; and that he would undertake them on the principle of "no cure, no pay" for a gross sum of $7 750 000, and in addition an annual payment of $100 000 for ten or twenty years, for the guaranteed maintenance of the works; but that under no circumstances would he consent to urge upon Congress the passage of any act in the premises.

This correspondence formed the basis for the bill pending before the late Congress (Senate Bill 1 652), which, however, failed to pass, although supported by the almost unanimous efforts of the people of Galveston and of the Texas delegation in Congress.

The issue therefore is as to what the result will be if the Government works be completed. They are not half done, and it is evident that the second or North Jetty must be built. A location for this, differing from the original location, was proposed in the last yearly report of the engineer-in-charge, it being upon a curved line gradually approaching the South Jetty, and leaving an opening of about 6 000 feet between them at the outer end. This has not yet been adopted, but it is estimated by Colonel Mansfield (Colonel Merrill's paper, page 227) that it may be built for $2 000 000, while the South Jetty can be raised above high water for $1 000 000 more.

Mr. Eads, on the other hand, says that the South Jetty is erroneously located, and he intimates that he would disregard it altogether in plan-

ning works to give a 30-foot depth. This is probably in consequence of the fact that instead of being located close to the thread of the main current, which issues from the "gorge," it is most of the way nearly a mile from it. To save expense, it has been built in shallow water, near the top of the subaqueous hill, instead of being near its bottom, and hence the issuing current is retarded by friction over the bed of the intervening space. This probably resulted from the fact that the original plans only contemplated obtaining 18 feet of water, and hence it was not deemed important to squeeze the current between the jetties in order to obtain as much scour as possible.

It seems to me, therefore, that in considering the difference between the estimate of Mr. Eads and that of the Government engineers, we should also consider the results which may be expected to follow. Mr. Eads proposed to guarantee 30 feet of water, not only over the bar, but along the city wharves, where it does not now exist, and as he offers to maintain the works for ten years, he would be compelled not only to make them permanent, but to protect them against the *teredo*, either by building them of concrete, or by thoroughly creosoting whatever wood he may use in them. For this he asks $7 750 000, payable as fast as the results are accomplished, and $100 000 a year for maintenance during ten years.

The estimate in Colonel Merrill's paper is $3 000 000 for the completion of the Government works. This, however, includes nothing for the inner harbor, but refers only to the jetties. These were planned originally to obtain 18 feet; but as only one has been built, the other may be relocated so as to produce greater results. It has lately been contemplated by the Government engineers that a depth of channel might be obtained across the bar of at least 25 feet. As this result can only be guaranteed by a contract, we shall have to be satisfied in this case with a strong probability.

The question therefore recurs: What depth of water will the Government works, if completed as suggested, give over Galveston Bar, and how permanent will they prove ?

I hope that the discussion upon Mr. Corthell's and Col. Merrill's papers will develop the opinions which are entertained upon this question.

Thus far the effect of the controversy has been to delay a much-needed public improvement. Galveston is the nearest seaport for a very large portion of the Southwest, comprising Texas, Kansas, Colorado and New Mexico, and the rapidly growing population of those States is already hampered for want of an adequate outlet to the sea. It is therefore important that the work should be resumed soon, and pushed to completion as fast as possible.

I thoroughly agree with Mr. Corthell in condemning the present congressional method of doling out (or withholding) annual appropria-

tions for carrying on important works which depend upon rapid progress for their success. Such a method is expensive, and sometimes disastrous, and should not be allowed to continue in Government works.

Capt. W. H. Bixby, M. Am. Soc. C. E.—Since the discussion on this paper has been opened, I feel in duty bound to add a few words against what I consider unjust criticisms and unjust reflections therein made upon the United States Corps of Engineers.

I do not think that the American Society of Civil Engineers, as a body, desires its Transactions used as a vehicle for personal adverse criticism of engineer upon engineer; and yet this criticism seems to have been one and the "final" object of Mr. Corthell's paper. I believe that, like myself, every Member of the Society read Mr. Corthell's article with brotherly pride and pleasure, as long as he confined himself to the praise of Mr. Eads and the description of the excellent work done by him. But I believe also that, like myself, a great many Members of the Society regretted the unnecessary disparagement of the work of other engineers, and especially of a corps which has been noted abroad and at home for its wise, thorough, conscientious and honest administration of the various hydrographic, topographic, geodetic and other civil engineering work entrusted to it by the Congress of the United States since 1802. I admire Mr. Eads for what he has done, and not for what other people may have left undone. Mr. Eads possesses a hard-earned and well-merited reputation for his excellent work upon the Mississippi Monitors, the St. Louis Bridge, and various other works, as well as upon the South Pass Jetties. But this is no reason why the work of the other engineers should be denied, or even ignored, by his historians. The "lessons" drawn by Mr. Corthell are in many cases unfair and unjust; and their unfairness can only be excused on the supposition that he is ignorant of what happened prior to 1874, and ignores what may happen after 1884.

The improvement of the bars of the Mississippi dates from 1837, when Mr. Eads was a small boy. The present method of improvement by "parallel jetties" at the mouth of the passes, combined with "the closure of lateral outlets, if necessary," was recommended to Congress by the United States Corps of Engineers in 1852, before Mr. Eads, as an engineer, had hardly had time to find out that the Mississippi River had a bar at its mouth. And the present successful results might have been obtained years before Mr. Eads took hold of the work if Congress had not handicapped the Corps of Engineers by specifying that the work must be done "by contract," and further by reducing the desired appropriations to amounts totally inadequate to the required work.

The Corps of Engineers are the servants of the people, not their masters. The recommendations of the Corps of Engineers have always been made according to their own best judgment. But their actual

work has always been restricted to what has been ordered by the people through its authorized mouthpiece, Congress.

The recommendations of the Corps of Engineers to Congress in 1852 were as follows:

1st. That the process of stirring up the bottom by suitable machinery should be tried.

2d. If this failed, that dredging by buckets should be tried.

3d. If both these modes failed, that parallel jetties should be constructed, five miles in length, at the mouth of the Southwest Pass, to be extended into the gulf annually as experience should show to be necessary.

4th. Should it then be needed, that the lateral outlets should be closed.

Finally, should all these fail, a ship canal might be resorted to.

And further, that there should be two jetties, one on each side of the channel, each jetty to be 14½ feet wide and composed of piles 2 feet apart.

In 1852, Congress voted $75 000 for this work, with the requirement that the work should be done by contract. This money was spent in dredging at the Southwest Pass, with results fully as successful, dollar for dollar, as is the dredging at present being done by Mr. Eads.* During the three following years, Congress failed to appropriate any money for the continuation of this work, and the channel gradually filled up again, just as it does to-day when Mr. Eads' dredges stop work,† and probably just as it will fill up in the next three years, if Congress fails to appropriate the money to pay for the continuation of such work.

In 1856, Congress voted $330 000 for opening, and keeping open for five years, the Southwest Pass and the Pass à l'Outre, with the requirement as before, that the work should be done by contract. In accordance with the recommendations of the Corps of Engineers, Messrs. Craig & Rightor, as contractors, made propositions to construct parallel or converging jetties on the bars and to combine this work with the closure of the minor passes. As in olden times it was found impossible to make good bricks without sufficient straw, so in Craig & Rightor's time it was found impossible to do good engineering work without sufficient money; and the $330 000 proved insufficient at that time to do work which has since been but partly done by Mr. Eads at an expense of several millions. The failure of the contractors to do the promised work, the requirement of Congress that the work should be done only by contract, and the War of 1861–65, put a stop for several years to any continuation of the system of improvement by jetties and dredging.

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* This foot-note added by Capt. Bixby subsequently to the closing of the discussion: This should be corrected to read "as by the dredging lately done by Mr. Eads."

† This foot-note added by Capt. Bixby subsequently to the closing of the discussion: This should be corrected to read "when Mr. Eads stops all work."

On the 25th of February, 1860, Captain A. A. Humphreys, Corps of Engineers, wrote as follows:

"The bars at the mouth of the Mississippi River are always forming, and a perpetual annual expenditure must be incurred to increase permanently the depth of water upon them. In this, all engineers who have written upon the subject agree. The appropriations for that object have, however, been given irregularly, and at intervals of several years; so that the deepening of the channels effected by one appropriation has been filled in long before the passage of the next. In fact, by the omission of the work of a single season, the bars may return to their original condition. To be of practical benefit to navigation, the depth of the channels must be permanently increased, a condition that could never be obtained under the system of appropriations heretofore followed.

"It may be remarked that the requirements of the appropriation act of 1856, that the work must be done by contract, has been one source of the defect of continuity in the operation, and the failure to maintain the increased depth after it was attained; for the failure of contractors to continue the deepening could only be ascertained after the shoaling, arising from neglect, had occurred. Then new contracts had to be entered into, and thus additional time was lost during the season for successful dredging and of commercial activity. But no plan whatever will prove of practical benefit to navigation unless a permanent fund be provided, untrammeled by restrictions as to the mode of expenditure, from which a sufficient sum annually can be relied upon for the continuous prosecution of the work after the channel has once been opened.

"A word upon the plan of deepening the channels by the construction of jetties on the bars, which Mr. Ellet states is in violation of the law controlling the formation of the bars.

"The experiments made by me demonstrate that this plan is based upon correct principles, and is in accordance with the law under which the bars are formed.

"The plan of jetties has not been tried at the mouth of the Mississippi to a sufficient extent to show whether it would be effectual or not, for the contractors merely built one insecure jetty of a single row of pile planks, about a mile long, on one side of the channel, whereas the Board of 1852 recommended two jetties, each 14½ feet wide, composed of piles 2 feet apart, one on each side of the channel, each 5 miles long, if stirring up the bottom and dredging should fail.

"It is probable, however, that the plan of stirring up the bottom is the more economical one of the two."

The Captain Humphreys of 1860 became General Humphreys, Chief of Engineers, a few years later, and was still Chief of the Corps of Engineers of the U. S. Army when Mr. Eads commenced operations upon the Mississippi Jetties. The recommendations of the Board of Engineers in 1852, and the statements of Captain Humphreys in 1860, are facts which must not be denied or ignored by any one endeavoring to draw just inferences as to the present work upon the South Pass Jetties.

The criticism of foreign engineers is often more to be depended upon than that of home engineers, because, while seeing all the good features of an engineering construction, they are free from party bias or prejudice. Let us see then what a French Engineer has to say about the South Pass Jetties (see remarks of M. Cadart, French Governmental

Civil Engineer, Ponts et Chaussées) in the November, 1884, number of *Annales des Ponts et Chaussées*).

While stating that the jetties have in general been successful, M. Cadart adds that their action is also felt over a fan-shaped area of more than one square mile in extent, and that for some time the bed of this area has been so raised by this action of the jetties that it is not yet possible to decide whether the improvement of the mouth of the river will be permanent or not.

As a mere matter of opinion, I think that there are quite a number of Members of this Society who, while patiently awaiting the development of the next ten years, are not yet prepared to deny that the jetties may still be undermined; that their foundations may still prove unstable; that the bar may still advance more rapidly toward the gulf; that the present channel may fill up rapidly as soon as the present dredging stops;* and that the ship canal may finally have to be resorted to as the only permanent method. It is to be hoped that the present generation, at least, will be spared such developments; but I think we are not as yet in a position to authoritatively state the contrary.

With the above facts and opinions before us, let us glance again at the lessons deduced by Mr. Corthell from work done at the South Pass between 1874 and 1884.

Mr. Corthell says :

"*First.*—Dispersion shoals—concentration deepens. The outlet theory is disproved," etc.

As Col. Merrill has already stated, all this was known, and its principles practiced with moderate success both abroad and in the United States prior to 1852.

Mr. Corthell adds:

"*Second.*—The flood volume of the river and the full volume of the tide are the powers which the engineer should bring to his aid. Jetties should be brought above the surface of the water at flood tide; outlets should not be left through them; and the jetties should be placed as near each other as the volume and current passing in and out show to be necessary for making a channel of adequate depth and width in the shortest possible time. Construction upon opposite principles has resulted in complete failure at Galveston and Charleston."

As regards the Mississippi outlets, this is just what the Corps of Engineers recommended in 1852, but what they could not do for lack of money.

As regards Galveston and Charleston, the facts do not justify the criticisms. Just as economy and the certainty of insufficient funds have caused many a man to build for himself a small and one-storied house, to which he expected to add enlargements and a second story when re-

* This foot-note added by Capt. Bixby subsequently to the closing of the discussion: This should be corrected to read "as soon as the present work is entirely completed."

quired by his needs and permitted by his funds, so have similar reasons caused the Corps of Engineers to first build submerged jetties, which they expected later to enlarge and raise above water as necessity should require and funds would permit. Mr. Corthell has seen at Galveston only the foundations and lower story of one-half of the house; has mistaken them for the whole house; and has ventured a sweeping condemnation of the unfinished structure and its architect. His mistake at Charleston is one of similar quality but less quantity.

Mr. Corthell adds:

"*Fifth.*—The works at the head of the passes have proved that willows and brush, used for simply slacking the current, will cause deposits and improve the contiguous channels. * * This system * * has been adopted by the Army Engineers throughout the Mississippi Valley, and it may be copied advantageously by the older nations of Europe, where the same materials have long been used for similar purposes, but with much less economy and benefit."

In behalf of classmates in European schools, and of brother members in European Societies of Engineers, I insist that full credit is due them for this whole system; that they have for years enunciated and practiced its principles; and that American engineers are not entitled to credit for anything else but the efficient way in which they have worked up its details. The tail may be a fine tail, but I object to its being allowed to wag the whole dog.

Mr. Corthell adds:

"*Finally.*—A lesson may be drawn from this matter concerning the proper mode of carrying on public engineering work. That lesson is, as it appears to the writer, that the Government should summon to its aid the best engineering talent within its reach, and should not give the general or exclusive charge of public improvements into the hands of engineers educated to conduct works of a totally different character.

"Let the doors be thrown open freely to all engineers fitted by education and experience for the service required.

"In connection with this lesson, it is a suggestive fact that no other civilized country puts its river and harbor improvements into the hands of other than civil engineers."

The results arrived at by Mr. Corthell are undoubtedly sound, but they either have no connection with the preceding premises, or they show a great ignorance as to the present and past history of Government engineering in the United States and abroad.

As has already been shown, the present method of improvement of the Passes of the Mississippi was recommended to the Government by the Corps of Engineers in 1852; and it is certainly unjust to blame the Engineer Corps because its recommendations were not followed.

The Government should certainly summon to its aid the best engineering talent within its reach; it has always done so in the past (even Mr. Corthell acknowledges this in his paper, where he says that

Members of the American Society of Civil Engineers were placed upon the Commission of 1874 for this very work of improvement of the Mississippi's mouth); it does so to-day; it will probably always do so in the future; not, however, by taking such work away from its own special Corps of Engineers; not by handing it over entirely to other engineers who have not been fitted by education and experience for the service required; but by calling upon specialists to serve as consulting engineers upon properly authorized engineer commissions, at a rate of pay proportioned to the value of their professional opinion, even though this might exceed that of a whole year's salary of a full Colonel or Chief of Engineers.

The Government certainly should not give the general or exclusive charge of public improvements into the hands of engineers educated to conduct works of a totally different character. It is for just exactly this reason that Congress, by Act 30th April, 1824, entrusted its road, canal, river and harbor works to the Corps of Engineers of its Army; and just exactly for this same reason that it has ever since then continued to repose the same trust in the same corps. Mr. Corthell appears to be ignorant of the following facts, viz.: That James McHenry, Secretary of War, by letter of 31st January, 1800, urged the re-establishment by law of a Military Academy and a Corps of Engineers, saying that "fortification is but a single branch of their profession;" "their utility extends to almost every department of war, and every description of general offices, besides embracing whatever respects public buildings, roads, bridges, canals and all such works of a civil nature;" that this Military Academy was finally established for the sole purpose of educating such engineers (Act of 16th March, 1802); that its Professor of Mathematics of 1805–10, F. R. Hassler, organized in 1816 our present coast survey system, a system which was more fully developed in 1843, and later by A. D. Bache, a Military Academy graduate of 1825; that its graduates have assisted in or have organized all the other grand topographical, geodetical and astronomical surveys of the United States; that until the late rapid development of the United States, its graduates have also individually planned and constructed almost all the great civil engineering works* of the country; that since then its graduates have been educated as before for exactly the same work; that at present only 5 per cent. of its graduates are allowed to enter the Corps of Engineers, showing a higher standard of requirement for such engineers; that its text-book of engineering (that of Professor Mahan, one of its own graduates) is to-day translated and known all over Europe, and is the basis of every American text-book of civil engineering; that the young engineer-graduates are first sent to the Headquarters of the Engineer Battalion for a two-years' post-graduate course of applied civil engineering, after which they serve

* This foot-note added by Captain Bixby subsequently to the closing of the discussion: This should be corrected to read "a large proportion of the great civil engineering works."

an apprenticeship under an older engineer before being put in direct charge of river and harbor work; and that, finally, each engineer-officer is subjected to a species of civil service examination, on all civil as well as military engineering subjects, at each step of promotion from a Lieutenant to a Major. Anybody and everybody who has carefully examined this matter knows the above to be facts, and therefore knows that the Government has placed the general charge of its public improvements in the hands of exactly those engineers whom it has, since 1802, been educating to conduct works of exactly that character.

It would certainly be an excellent thing if the doors of the Government Civil Engineer Service could be thrown open freely to all engineers fitted by education and experience for the service required, provided that they are at the same time to be kept closed against all others not fitted for such service. Is there any better method of so doing than the method now in practice? The Military Academy at West Point is opened freely to all young men from all quarters of the United States, with no restriction as to race, color or previous condition of servitude. The Engineer Corps is open freely to any Military Academy cadet whose high-class standing shows a probable capability for the duties to be afterwards required from him. Can any known method be freer or fairer than this, and is there any other known and tested method which guarantees to the Government such a constant succession of reliable and efficient officers? I know a great number of eminent engineers, who are Members of the American Society of Civil Engineers, whose experience is varied, whose reputation as engineers is beyond reproach, and who would be ornaments to any corps of engineers—most of them are at least as old as Majors in the Engineer Corps—yet I doubt much whether any of them have had any experience in the details of administration and disbursements such as Congress requires of its engineer-officers, and I feel very sure that very few of these men would be willing to devote any large amount of their time to such details; very few, if any of these men would be willing to exchange their permanent homes, their various outside religious, social, political and business interests, and their proportionately large net incomes, for the constantly changing homes, the isolation from all outside interests, and the proportionately small net incomes of a Major or Colonel of Engineers. In short, those Members of the American Society of Engineers who are fitted by education and experience for the service required by Congress from its Government Engineer-Officers, are exactly the ones who would not be willing to devote their whole time and energy to the Government for any such salary as the Government now pays for such work.

I do not know that any civilized country puts its river and harbor improvements into the hands of other than its civil engineers. I think that I have shown already that the United States does not do so. At the same time it is a suggestive fact that one of the finest corps of organized gov-

ernment civil engineers in Europe (that of France) has always been recruited exactly as the United States is at present recruiting its own Government Engineers. In order to become a member of the French Corps of Government Civil Engineers, viz., of the Corps des Ponts et Chaussées, a man must first enter the École Polytechnique (their military academy), as in the United States he must first enter the West Point Military Academy; he must there follow a course of study from which that of the first three years at West Point has been copied; after this, if he stands near the head of his class, he is allowed to follow a two-year course of study at the post-graduate school of Ponts et Chaussées, a course equally comprehensive with that of the last year at West Point combined with the post-graduate course at the headquarters of the battalion of engineers; after this he is admitted to the corps of engineers, being generally assigned at first to duty under some older officer, exactly as is done in the United States. With rare exceptions (those of assistant engineers who have served as such continuously and faithfully for many years) no one can possibly become a government civil engineer in France except by following the route above described. The many technological schools of France graduate ordinary civil engineers, but no government civil engineers; the French government, since the time of Napoleon, having always required its government civil engineers to prove their willingness, ability and fitness for their position by an actual successful passage through the training schools of both its Military Academy (the École Polytechnique) and its post-graduate Government Engineer School of Bridges and Highways (École des Ponts et Chaussées). As a graduate of the latter school, and also as a Member of the French Society of Engineers, and having spent two years amongst both these classes of engineers, I feel that it is not asking too much of the American Society of Civil Engineers to ask them to accept these statements as facts.

In another of the finest organized corps of government civil engineers in the world (that of Prussia), the executive organization is almost the same as that of the United States. In Prussia, as in the United States, the water-ways are divided into districts, each under the charge of a single government engineer, who is held responsible for the work done by him, and who is supervised more or less by his superiors. In the United States this supervision consists in the criticism and recommendation of a Supervising Engineer (one to five or six districts), of the Chief of Engineers, increased in important cases by that of the Government Board of Engineers, and in more important cases by that of a Board of Engineering Specialists selected generally from amongst the Members of the American Society of Civil Engineers. In Prussia, the method of appointment to this corps of engineers differs somewhat from that in the United States, but there is no proof that the former is better than the latter; and the probability is very great that the system used in

monarchical Prussia, if applied to republican America, could not guarantee to the country such a continued succession of capable Government Engineers as does the present system. In the United States, Congress chooses to retain in its own hands the original decision as to what works are to be improved and as to the proportion of money to be spent on each; and the Corps of Government Engineers is charged only with the final execution of this work. In Prussia, the government engineers are given much more power and authority than in America, and can, therefore, select much more successfully the works which best merit improvement, and can carry out more successfully the improvement finally decided upon.

An important lesson that Mr. Corthell might have added here, is that Government civil engineering work cannot be carried on thoroughly, successfully, and economically by any corps of engineers as long as the proper execution of this work depends upon discontinuous, spasmodic and inadequate appropriations.

J. F. Le Baron, M. Am. Soc. C. E.—The results accomplished by the construction of the South Pass Jetties are so patent to the world that they cannot be disputed. The fact is the Mississippi Valley is open to the commerce of the world, untrammeled by bar, and unvexed by extortionate tug charges or delays.

The report of General Bussey, President of the New Orleans Cotton Exchange, as quoted by Mr. Corthell, is conclusive. He says that now 95 000 bushels of grain is an ordinary cargo, and 144 000 bushels is sometimes taken; whereas, before the jetties were constructed, 20 000 bushels was a large load. This shows an increase of carrying capacity of between 400 and 700 per cent. In the face of this glorious result, the remarks of Col. Merrill, referring to the fact that the first contract with Mr. Eads, requiring a channel 26 feet deep and 300 feet wide, had never been fulfilled, seem rather finical. The width is wide enough and the depth deep enough for all practical or theoretical purposes, and is now ten feet wider than the later Act of Congress requires.

We must not lose sight of the fact that this result must be compared with the results achieved by the army engineers previously.

"Thirty-seven years ago," said the Hon. Carl Schurz, in a speech delivered in the United States Senate, "the Engineer Department of the Army took the matter in hand, and for thirty-seven years they have been planning and reporting upon the matter, and scratching and scraping at the mouth of the Mississippi, and to-day the depth of water is no greater than it was then."

It remained for an "outsider" to achieve the glorious results that will render the name of Eads famous for all time, and that, too, in the face of the bitterest opposition, as is well known.

So much for results. Now for the lessons.

It seems to me that one of the plainest teachings of the South Pass Jetties is, that such works should, when once commenced, be pushed forward with all the rapidity possible, and I think that to the rapidity of his construction at South Pass, Mr. Eads owes a great part of his success.

It has been said of Napoleon that he dazzled by the rapidity of his movements. In the same way Mr. Eads has dazzled the world by the rapidity and success of his efforts at the mouth of the Mississippi, and before the river could form a new regimen, the works of contraction were completed, and the river was conquered and compelled to follow the path laid out for it.

The works at the head of the passes show the importance of conservation of the river forces, and the fallacy of the "outlet theory."

Another important lesson is that a river will, with the proper assistance, obtain what it will maintain in the way of cross section.

The retardation of bar advance, by reason of the concentration of volume, and increased slope and velocity of the river current, does not appear to me to be so plainly proved as Mr. Corthell seems to believe.

We have as much sediment brought down by the river; and in fact more, by reason of the greatly increased volume of discharge. That this is carried further out to sea by the increased force of the current, is evident; but in the absence of any littoral current, this immense amount of sediment must be deposited in the fan-shaped area beyond the seaward ends of the jetties. How far beyond depends upon the velocity. If this sediment is held in suspension until depths of 600 or 1 000 feet are reached, before it is dropped by the current, then we may safely dismiss the question of bar advance for a century or two; or if a littoral current exists that will take this sediment and sweep it to one side, before or as soon as it is deposited, then we are safe. But this is not shown by actual examinations. The immense amount of 172 800 000 cubic feet of sediment that Mr. Corthell states is carried out every 24 hours of flood discharge, must in time raise the bottom of the gulf wherever deposited.

The portion carried out through the South Pass is not directly given by Mr. Corthell; but supposing it to be one-fifth, it is then equal to an area over a mile square raised one foot every 24 hours of high water.

What is the ultimate disposal of this immense amount of material? Mr. Corthell quotes Mr. Bayley to show that there is a littoral current, but does not state what its velocity is, or if it has ever been measured. In the absence of this data, and not knowing how far out this sediment is carried, we are left somewhat in the dark as to the ultimate elevation of the bottom, and the consequent bar advance. Estimating it by geological ages, it would appear certain that some time in the future the jetties would have to be extended.

Turning now to another part of Mr. Corthell's paper, that which refers

to Galveston and Charleston Harbors, we find an entirely different problem presented. The bars of these harbors are caused, not by sedimentary deposits, but by the littoral action of the waves abrading a sandy coast and attempting to perfect a littoral cordon, which is feebly resisted by the discharge of the ebb tides and the emptying of a land-locked lagoon, whose level is but slightly above that of the gulf, and varying at different seasons.

It seems hardly proper to compare the two, and it is certainly unfair to compare the results obtained by the army engineers at these places, working with small annual appropriations, with the results obtained by Mr. Eads at South Pass, working continuously, and with abundant means at his disposal. I very much doubt if Mr. Eads could have achieved the success he undoubtedly has at South Pass if he had been restricted to $200 000 or $300 000 per year; and the same remarks apply to Charleston. The works at Charleston are yet far from completion, as originally planned; and I believe, with Colonel Merrill, that on the works at Charleston nothing has been done but what would have had to be done in any event. Until the jetties at Charleston shall be raised somewhere near their estimated height, and brought nearer to completion, I think it would be proper to withhold criticisms of their efficiency. It is like fencing in an acre of ground to keep in a herd of cattle. While the fence is incomplete on one side, if it is only for a yard, the herd is not restrained from escaping. We may build ever so costly and high a wall on three sides, but until the fourth side is completed we achieve no result. The jetties at Charleston will undoubtedly serve the purpose intended by General Gillmore in preserving a channel from the littoral wave-action and wash of the sands, when completed.

As to Galveston, it appears that Mr. Corthell is sustained in his criticism as to the width between the jetties by the chart published in the Annual Report of the Chief of Engineers United States Army, for 1880, on which chart the North Jetty is shown 18 000 feet from the South Jetty, and is marked at its shore end by a line of gabionades set as part of the jetty. And in the report for 1882 we see an opening left near the shore in the South Jetty; but from the report of Colonel Mansfield we learn that this gap was being filled by a continuous jetty. It could only be considered a temporary gap. It was not too late to fill it, and I do not see that any harm was done by temporarily leaving it open. If it was intended as an inlet for the flood tide, it was, in my opinion, unwise and faulty engineering.

In his discussion, Colonel Merrill quotes Colonel Mansfield as stating that the location of the North Jetty was undetermined; but upon the chart in the report for 1880, before referred to, this jetty is located, and reference is made to it in the report. The Bolivar channel is only about 3 300 feet wide between the 20-foot curves, and it would appear that the

location of the jetties 18 000 feet apart was very excessive and unwarrantable. We find, however, that this jetty was discontinued, and it does not appear on the chart for 1882. I have referred to this simply to sustain Mr. Corthell in his assertion.

It appears to me improper to compare completed works with uncompleted ones; but still we may criticise them, and may question the wisdom of the location of the jetties at Galveston, and the application of the money expended on one jetty alone; but we can hardly assume that, even with a proper location and insufficient funds, any different result would have been obtained.

The slowness and smallness of congressional appropriations for river and harbor works is a fruitful source of expense, and is extravagant in the highest degree, retarding the development of the country and her commerce. So much so that public patience is exhausted, and often seeks relief in personal efforts. I am now engaged in perfecting plans for improving a river mouth by jetties in Florida, which the people of the country propose to build without asking aid from the Government, owing to the reasons stated.

In regard to the last part of Mr. Corthell's paper, which refers to the exclusive employment of army officers in charge of civil works of this description, I agree entirely with the author of the paper. Colonel Merrill refers to the fact that United States Army Engineers are educated as civil engineers. This no one disputes, nor their ability as civil engineers; but it is manifestly unjust to employ any persons on an equality, and deny them all opportunity or right of advancement or promotion; and if not employed on an equality the unjustness is greater, in a free country, under a republican form of Government. Such, however, is the condition of civil engineers on Government works at present.

Gen. Q. A. Gillmore, M. Am. Soc. C. E.—I wish to correct one inaccuracy in the paper by Mr. Corthell, with respect to the jetties now in course of construction at Charleston, S. C. The paper states that there is 2 feet less water on the bar than when the jetties were begun. This is not so. At the time the paper was read, the work done at the Charleston Bar comprised little more than the apron foundations of the jetties. That of the North Jetty reached beyond the crest of the bar a little, while that of the South Jetty stopped short of it, and the amount of contraction had reached only a small fraction of what the project contemplates. A section of the water-way between the jetties, taken at the point where the foundations of the South Jetty stopped, showed an enlargement and an increase of mean depth quite as great as could be expected from the slight concentration of water at that point, and indicated quite clearly that the completion of the works will result in an improved channel fully up to the estimated depths, and that these

depths can be increased by raising and extending the jetties, without carrying those portions crossing the deep water near the shore up to the water level.

The project is only a tentative one, but the limit of its beneficial effects will be found to be quite up to the present requirements of commerce.

I have never thought that even the sea-ends of the jetties—those portions located on the bar—would need to be carried up to high-water level, but if there will be any advantage in giving them that height, it will, of course, be done.

There is another statement in Mr. Corthell's paper which, I am sure, he will wish to correct, where it is said that work on the jetties has been going on for nearly 10 years, or words to that effect. So far from this being the case, only 5½ years had elapsed, at the time he read his paper, since the works were first started, and during that interval there had been one year when no appropriation at all was made for the work, and there had been frequent suspensions for want of money. Indeed, all the work done could easily have been accomplished in two years if funds had been available.

I append to this a communication addressed by me to Hon. S. Y. Tupper, of Charleston, on the 22d of January last.

CHARLESTON, S. C., January 22d, 1885.

Hon. S. Y. TUPPER,

Chairman Committee on Harbor Improvements of Charleston Chamber of Commerce.

DEAR SIR,—Referring to our recent interview, I wish to express my satisfaction with the progress of work on the Charleston Jetties under the present contract, and the beneficial results in immediate prospect.

Capt. Bailey informs me that the survey just completed indicates quite plainly that increased scour on the bar between the jetties has been developed.

Although the concentration of water upon the bar secured by the jetties up to the present time is very small in comparison with what is intended and with what the plan requires, both works being as yet little more than foundation courses, and the South Jetty not having yet reached the crest of the bar by about 1 000 feet, I am confident that, even in their present condition, the works are able to maintain a much deeper channel than that which exists between them, and that the time to begin the dredging contemplated in the original estimate has arrived. Proposals for dredging have, therefore, been invited, the object being to aid the scouring power of the accelerated current.

I wish to reaffirm my confidence in the ability of these works, when completed, to maintain a deep navigable channel across the bar. I believe a depth of 28 to 30 feet to be easily attainable.

Adverse criticisms on the efficacy of submerged jetties have no application to the Charleston works. The latter are strictly tentative in character, and their distance apart was planned with the special object

of reserving entire freedom of choice with respect to their ultimate height.

Their crests can, of course, be carried up to the height necessary to maintain the desired channel depth between them. Where they cross the bar they are half a mile apart, equal to about one-half of the width of the throat of the harbor between Forts Sumter and Moultrie, where the channel depths are 75 to 80 feet at high water.

Omitting engineering technicalities, two significant questions may be popularly stated as follows:

1st. If the flow between Sumter and Moultrie maintains a channel depth of 75 feet where the width is 1 mile, what portion of that flow will be required between the jetties where the width is only half a mile, in order to maintain a depth of 28 or 30 feet? The answer suggests itself that not all the water flowing past Fort Sumter will be needed between the jetties. The plan, therefore, allows part of it to pass over their tops, thereby lessening the cost of the works more than one-half.

2d. If the flow between Sumter and Moultrie maintains a depth of 75 feet where the width is 1 mile, what depth would the same volume of flow maintain if it all passed between the jetties where the width is only half a mile? The answer is again evident, that under these conditions, the depth scoured out would be excessive, possibly endangering the stability of the works themselves, and transporting a large body of sand to the seaward slope of the bar beyond the end of the jetties. This would introduce serious complications in the way of new shoals and a new bar. It has been one of the leading objects of the project to avoid such a contingency.

The principles suggested in the foregoing answers are neither new nor novel. They not only rest on long experience and observation, but are easily demonstrable by approved methods and formulas. We need not go far for numerous and well-defined illustrations—two or three that are near home will be mentioned.

At the mouth of St. John's River, where the tide rises and falls about 5 feet, the low jetties planned on the principles applied to the Charleston works, and not yet half finished, have produced a marked improvement in the gap between them. A channel has there been established by scour alone, of twice the depth existing there six months ago.

In Savannah Harbor, where there is about 7 feet rise and fall of tide, the Fig Island Jetty, 1 mile long, with its crest fully 4 feet below high water, maintains a navigable channel 19 feet deep at high water, through a shoal upon which there was only 11 feet before the improvement was begun. Further down the harbor, at the head of Elba Island and at the Upper Flats, the low jetties, having their crests about 3 feet below high water, are continuously maintaining navigable channels 20 to 23 feet deep, through shoals where only 16 feet could be carried at high water before any improvement was made by works or by dredging. These are points where the jetties have been completed a year or more, or long enough to produce a substantial approach to maximum results. At other points where the jetties were finished more recently, the scouring is now in progress. In some cases the movement of the sand is so tardy that the deepening process has to be hastened or aided by other means, and this is the object of the dredging which will be commenced in Savannah Harbor in a few days.

It goes almost without saying that the Charleston Jetties should not excavate a channel of unnecessary depth between them, with the certain

result that the excess of material removed will be deposited outside where the natural depths are very moderate and the slope of the bottom exceptionally flat, and where we must depend for its removal upon the intermittent and irregular littoral current produced by winds. Where interests of great magnitude are at stake, ordinary prudence suggests that there should be a liberal factor of safety. This is provided in the present project by reserving the power to restrict the volume of flow and the amount of scour between the jetties.

It has been the subject of profound regret on my part that the importance of this great national work has not been recognized in Congress by liberal appropriations. Frequent stoppages and increase in the ultimate cost of the work have been the results of this policy.

I invite atttention to my annual reports for a more lengthy expression of my views on this point. Q. A. GILLMORE.

E. L. CORTHELL, M. Am. Soc. C. E.—The three subjects treated by Col. Merrill in his discussion are: The South Pass Jetties; Galveston Harbor Improvements; and Civil and Military Engineers on Government works. The first two subjects have been so fully discussed by Mr. James B. Eads, that it is unnecessary for the author to recur to them. He will therefore confine his discussion to the third subject.

Col. Merrill, referring to the author's statement regarding the manner of conducting national public works in this and other countries, has taken exceptions to his views and attempted to correct his statements. The subject is too important to be lightly passed over, and his criticisms require discussion.

We may profit by the example of the other civilized nations referred to. There, as here, engineers are divided by education and practice into civil and military. The governments do not employ military engineers, as a general rule, on their public works, except for certain very limited geodetic work bearing upon military problems and for military purposes.

The following is from a letter by a leading Member of this Society in France, Mr. Ernest Pontzen:

"All public works (civil engineering works) executed by the Government are conducted by the Corps of Ponts et Chaussées. The Corps of Ponts et Chaussées, like the Corps of Mines, is under the Minister of Public Works.

"The Corps of Ponts et Chaussées is at present composed of forty-eight 'General Inspectors,' in two classes; two hundred and fifty-eight 'Engineers-in-Chief,' in two classes; and three hundred and thirty-one 'Ordinary' engineers in three classes, being a total of six hundred and thirty-seven engineers of the various classes. Besides these engineers, the Corps also has lower officers in a greater number, called 'Conductors.'

"The young man who intends to become an engineer, tries generally to be admitted to the Polytechnic School (a Government institute). Among one thousand candidates, only about one hundred and ninety or two hundred are admitted to this school. The best pupils generally choose

the Corps of Mines or the Corps of Ponts et Chaussées. The remainder, after being two years in the Polytechnic Schools go into military service. The engineering schools into which the first pupils enter after leaving the Polytechnic School have a three years' course. When a young man has obtained, in this way, his admission into this school, he comes out as 'Ordinary' Engineer of Ponts et Chaussées, or of Mines. These are civil and not military positions. An engineer who has pursued his way through these schools and obtained his rank, has to remain ten years, often ten or fifteen a 'Conductor,' and then to pass an extraordinary examination in order to be titled 'Ordinary Engineer.' All works executed by the Government are conducted by such Engineers of the Ponts et Chaussées. 'Ordinary' engineer is 'Resident' engineer. The Engineer-in-Chief has three or more ordinary engineers under his control, and a General Inspector has ten "Engineers-in-Chief.' The General Inspectors in Committee decide on all works executed in France. It is called the 'Council General of Ponts et Chaussées.'

"Since the beginning of railways, the highest positions on railroads are filled by engineers of Ponts et Chaussées, who obtain general permission to enter on private business. Latterly the Minister of Public Works has selected civil engineers to be members of ten committees in the Department of Public Works in order to give satisfaction to the civil engineers; that is, those educated in private schools who objected to the monopoly of the Corps of Ponts et Chausées. I, myself, am now a member of the Committee on Railroads under the Minister. Some propositions are now before the Chambers to open still wider the doors of the Corps, and probably in the future, ten per cent. of the Corps of Ponts et Chaussées will be taken from among the civil engineers educated in private schools and practiced in private works.

"The military engineers are used only for building the forts and other military constructions, like the navy engineers, who build the naval constructions; but civil engineers are often contractors for these works."

It will be seen from this letter that the public works of France are conducted under a Minister of Public Works; that civil engineers are specially educated for these works; that many years of actual practice are required for admittance even to the class of "ordinary engineers;" that they are graduated as civil engineers, practice as such, and are employed only as such; but that large numbers of other prominent and experienced civil engineers, whose education is obtained in private schools, are admitted to places of the highest honor and responsibility in designing and constructing government civil works; in other words, using the expression in the author's paper, the government summons to its aid the best engineering talent within its reach.

In Prussia the system of public works is much the same as in France. The country is divided into river and harbor districts, with a chief engineer at the head of each district. These districts are subdivided into lesser districts, with a resident engineer over each. These resident engineers are provided with two classes of assistants: first, those who have passed the double examination required; and, second, those who have passed only one of these examinations. These two latter classes

The following letter has been received from Mr. E. L. Corthell, M. Am. Soc. C. E., and is published as requested.

34 Nassau St., New York,
July 30, 1886.

John Bogart, Esq.,
Secretary Am. Soc. C. E.

Dear Sir.—Partly through the inadvertence of a type-writer, partly through my misapprehension of the meaning of French terms used, and partly for the sake of brevity, a letter of Mr. Ernest Pontzen, from which I quoted in the discussion of my paper on "The South Pass Jetties" does not present the ideas as he intended to convey them, and he requests me to make the necessary corrections.

Will you kindly have them made in the next publication of the Transactions of the Society.

As Mr. Pontzen is a Member of the Society and was not able to be present to take part in the discussion, it is a simple matter of justice to him that an erroneous presentation of his views should be removed, which I hasten to do.

I am, yours truly,
E. L. Corthell.

TRANSACTIONS AMERICAN SOCIETY OF CIVIL ENGINEERS.

ERRATA.

Volume XV, April, 1886. Discussion by E. L. Corthell on the South Pass Jetties.

Page 270, 7th line.—Insert the word "not" after the words "an engineer who has." The sentence will then read: "An engineer who has not pursued his way through these schools and obtained his rank, has to remain ten years, often ten or fifteen a 'Conductor,' and then to pass an extraordinary examination in order to be titled 'Ordinary Engineer.'"

Same page, 28th line.—After the words "from among the," insert the word "Conductors" in place of the words "Civil Engineers."

Same page.—After the 29th line insert the following paragraph (the final paragraph remaining as printed):

"It must be confessed that the high rank of the Corps des Ponts et Chaussées must be attributed to the system of admission through the best schools and admitting only the best pupils. This system is a democratic system, its positions being given not as a privilege to rich or noble people, but to the most able young men. Nevertheless, I believe that the doors are not enough open to such engineers as succeed in educating themselves in other ways with the intention of obtaining governmental positions. In France the tendency to be an employé of the government is a general one. I believe it is not so in the United States, and it proves that your people have more feeling for independence, and it is better for you."

work for the government or private corporations, or both. According to their length of service on government works they advance to positions of inspectors and resident engineers. In case of large rivers, such as the Rhine, Elbe, Oder and Vistula, the whole river constitutes one district, whose chief engineer has charge of all improvements on that river. The principal office of River and Harbor Works is a bureau in the Department of Public Works.

The above description of the system practiced in Prussia, is abstracted from a paper read before a German Society of Civil Engineers by a prominent member of the Prussian Corps of Civil Engineers, who was for several years a resident of this country as engineer, attached to the Imperial German Legation at Washington. Comparing this system with our own, it is under a Minister of Public Works instead of the Secretary of War; and the officials in charge of the works are state *civil* engineers instead of *military* engineers.

A translation of the paper referred to was read before the Boston Society of Civil Engineers, January 14th, 1885, by Mr. Clemens Herschel, Member of this Society, who states, on page 16 of the paper, "From what I know of public works elsewhere, in Europe—France, Italy, Holland and other countries—I believe their organization for the construction of public works to be very similar to the comprehensive system above described."

Confirmatory of the preceding statements, the late Dr. S. Hagen, of Berlin, in a letter dated July 12th, 1880, stated: "The co-operation of superior military officers and officers of the engineer corps on our civil constructions or structures is entirely excluded, except where works of fortification are connected with them. All civil works of the government are carried on under a Minister of Public Works. The same relations exist in the other German States, and you will see that in France, England and the Netherlands it is the same."

Col. Merrill says: "Great Britain makes exactly the same use of her military engineers as does the United States."

He says just enough to give the impression that the public works of Great Britain, as far as relates to its rivers and harbors, are conducted as they are here, by military engineers.

The following letter recently received by the author from Mr. James Forrest, Secretary of the Institution of Civil Engineers of Great Britain, through Sir Charles A. Hartley, will disabuse the minds of the Members of the Society from the impression sought to be conveyed by Col. Merrill.

"The Institution of Civil Engineers,
"25 Great George street, Westminster, S. W.,
"24th February, 1885.

"Sir Charles A. Hartley, K. C. M. G.

"My dear Sir Charles,—In reply to Mr. Corthell's inquiries, through you, I have to say that there can be no parallel between the employment of civil (*i. e.*, civilian) engineers in the United States and in this

kingdom, for the simple reason that few works are undertaken here by the Government, except dockyards and harbors of refuge; the latter mainly because they would not pay commercially and because they afford a field for convict labour.

"There are no Government C. E.'s; the only engineers that can come under that category are Royal (or military) Engineers.

"These officers frequently obtain civil appointment, but rarely to design or carry out works.

"Rivers and harbors here are under Boards or Trusts, who go into the open market and secure the services of any engineers they may choose to select.

"I think the above nearly exhausts Mr. Corthell's questions. I may add that the only approach to a Government Engineering College in this country is the Royal Engineering College at Cooper's Hill, near London. That was created some years ago for the purpose of systematically training young men for the Public Works Department of the Government of India. Even in that great dependency of the British Empire, all the leading railways have been executed by private companies with engineers selected by themselves, without let or hindrance from the Government. * * * * *

"I remain,

"Yours very sincerely,

"JAMES FORREST,

"*Secretary.*"

A brief description should now be given of the anomalous system of the United States, which has by no means kept pace with the wonderful growth of the country and its important and increasing demands upon civil engineering skill.

It seemed necessary, at first, that the graduates of what was then the only school in the country that taught engineering, even though it was military engineering, should be selected to take charge of the small beginnings of Government River and Harbor Works, especially as the country was at peace and the officers could be spared occasionally from their military duties.

In addition to the requirements of our Government for civil engineering talent, there has been a demand for educated and skilled civil engineers in the great private enterprises of the country—roads, canals, railroads, bridges, tunnels, mines and water-works, with all their varied and associated constructions. The educators of the country have not been slow to appreciate and meet the necessity for educated civil engineers. Fifty years ago, West Point Academy probably taught more engineering than any other school; but there is no doubt that to-day it teaches much less of it than many private schools and colleges that might be mentioned. Nor is this all that might be said of our civil engineering schools. They are far in advance of similar schools in Europe.

Professor E. A. Fuertes, Member of this Society, in a recent letter to the author, states the following:

"After a detailed study of these questions on the ground, all over Europe, and under the best auspices, it is my belief that, all things

taken together, there are one-half dozen schools in this country that can turn out better material for making civil engineers than any school I saw in Europe, excepting one in Germany and one in France. And, comparing the attainments of West Point men with Troy, Ann Arbor or Cornell men, they may be said to be quite inferior. In the first place, a young man entering this department at Cornell requires more than one year more of mental training and higher knowledge than to enter West Point. Then he is put through a very severe course of training here, exclusively dedicated to make him an engineering student; while a glance at our respective courses of study will show that more than one-half the time of the West Point graduate is taken up with military work. Therefore it may be said that, in reference to civil engineering aptitude, the Cornell man is to the West Point man as 5 is to 2, all other things being equal."

Probably there has been no retrogression at West Point; but there has been a great advance in all the other schools, to meet the demand for thoroughly educated civil engineers.

At West Point the first fifth in each class are assigned on graduation to the Engineer Corps of the Army, and their excellence in all the military qualifications of their profession is probably unequaled. They understand from books and from capable instructors the methods of constructing the works necessary for offensive and defensive warfare. They are, however, after remaining a while in a school of *military practice*, placed in charge of designing and controlling the civil works of the Government. Under our present system they have, therefore, many of them, two classes of duties to perform, those which appertain to their profession, and those which belong to civil engineering; for, under the chief of the military engineers are placed all the extensive and important river and harbor works of the Government, on which is expended annually from ten to fifteen million dollars. The civil assistants of these military engineers make the surveys of the harbors and rivers, map out the hydrographic and topographic work, often design the location and construction of the works, are familiar with all the conditions on which a proper design depends, and make the estimates of the cost of the works.

There are about sixty officers in charge of the numerous public works of the Government, and there are under them about three hundred civil assistant engineers, exclusive of draughtsmen, surveyors, inspectors, superintendents, etc.

The civil engineers find the doors of advance closed against them, no matter how well educated or skillful they may be, or how long their connection with the work may continue. It can hardly be expected that men of parts, educated and experienced in the works of their profession, should remain long in the employ of the Government under such circumstances. To their own regret, and their country's loss, they seek employment elsewhere. In civil life, engineers are selected for their special fitness by education and experience for the work in hand; but so few are the officers, and so many and varied are the works, both military and civil, which the Government engineer is obliged to perform,

that this discrimination cannot be made on the public works of the Government; and it is often the case that some officer who may have been, since his graduation, employed on a boundary survey, is detailed to deepen a harbor, improve a river, or design and execute water-works.

There exist very cogent reasons for some change in the present system of National public works from their very inception. These reasons have, on several occasions, been presented to Congress.

The author quotes from a "Memorial Relative to the Proposed Mississippi Improvement Commission," presented to Congress some years since, and signed by thirty influential Members of this Society:

"The work proposed is of vast scope and importance, and likely to involve great expenditure. The highest authorities differ as to the principles that are applicable in the premises. The regulation of the drainage of half the continent is a problem of the most serious character, and the country should have the very best advice that can be had.

"It is true that a commission of few members, belonging to the same school of practice, will more readily arrive at unanimous recommendations than one composed of a greater number of members, of varied experience and different qualities of mind; but the conclusions of the latter would, probably, be the safer.

"Although the construction of works done at the charge of the general Government has long been in the hands of the army engineers, it is no less true that the works of public utility executed by civil engineers far exceed the former in magnitude and importance, and it appears unwise that the country should be deprived of the benefit of the experience acquired by the civil branch of the profession.

"For these reasons the undersigned would respectfully suggest that the number of the commission should not be less than seven, comprising three officers of the Engineer Corps of the Army, three eminent civil engineers, and one member of the United States Coast Survey, and that the efficiency of the commission would not suffer, by reason of numbers, if two or more persons from civil life were added to it, one familiar with the navigation of the river, and the other with the lands subject to overflow."

Another memorial, drawn up also about four years since, is a valuable contribution to this subject, and as it so fully sets forth the reasons for the employment by the Government of civil engineers in its more important works, we give its introductory clauses.

"A memorial relative to the establishment of a River and Harbor Improvement Commission.

"To the Senate and House of Representatives of the United States, in Congress assembled:

"The undersigned, citizens of the United States, Members of the American Society of Civil Engineers, and constantly engaged in works of civil engineering, beg leave, in view of the approaching consideration by your honorable bodies of the 'Bill for the Improvement of Rivers and Harbors in the United States,' respectfully to present for your consideration the following facts:

"The present system under which the appropriations made by the bill just mentioned are expended has many disadvantages, not the least among which is the exclusion, from any but the subordinate positions on the works, of the Civil Engineers of the United States. Many of these

are eminent in their profession, and accustomed to carry on with success and economy works of greater magnitude than any contemplated under the annual provisions of the 'River and Harbor Bill.' The country by their exclusion, and the want of their counsel and advice loses both in money and time.

"The vast importance to commerce, and, consequently, to the country at large, of the improvement of our harbors and navigable streams, needs no discussion. This being universally conceded, the Government is entitled in the prosecution of such works, to have the benefit of the most skilled and intelligent labor. This, under the present system, it does not have.

"The public works under the 'River and Harbor Bill' have heretofore been, and are now, carried on under the direction of the Chief of Engineers of the U. S. Army, by officers of the Corps. So inadequate in point of number, however, is the force available for the works, that many civilians are and have been employed in subordinate positions; nominally under the direction of the officers of engineers in charge, but excluded from all share either in the responsibility of the work or in the reputation that might be derived therefrom. The country is, therefore, practically deprived of the benefits arising from the knowledge, experience and professional skill of many of the most eminent civilian engineers; who are, naturally, unwilling to accept subordinate compensation for their labors, either in money or reputation. Thus the works are delayed, and the burden of additional expense is thrown upon the people."

To sum up these facts and arguments:

First.—By an accidental precedent the River and Harbor Works of the Government, were placed in the exclusive charge of the Military Engineers.

Second.—The large number of civil assistants employed by these military engineers hold a very subordinate position, unworthy of their attainments, and without prospect of advancement or of reputation.

Third.—This anomaly prevents the Government from obtaining the services of the best civil engineers, and debars them from work to which their education and skill particularly adapt and entitle them.

Fourth.—Our commercial interests have increased to such an extent the number and the importance of the Government public works that they cannot well be superintended by the military engineers in connection with their other duties.

Fifth.—There should be some legislation by Congress which should separate the military from the civil works, or at least establish a system that should not subordinate the civil to the military. We have only begun these great public works. They will in a few years assume proportions that cannot be conceived of now; and it is necessary that some system be adopted to meet the future wants of the Government and the necessity which will then exist, if it does not now, for the employment of the best civil engineering talent.

Therefore, referring again to the paper on the South Pass Jetties, now under discussion, the final issue taken with the author by Colonel

Merrill is in the final lesson of this paper. He has, however, failed to take the lesson as given, either in its letter or spirit. The lesson the author gave was this:

"Finally, a lesson may be drawn from this matter concerning the proper mode of carrying on public engineering works. That lesson is, as it appears to the writer, that the Government should summon to its aid the best engineering talent within its reach, and should not give the general or exclusive charge of public improvements into the hands of engineers educated to conduct works of a totally different character."

James B. Eads, M. Am. Soc. C. E.—Col. Merrill's paper purports to be a continuation of the discussion of Mr. Corthell's paper on the South Pass Jetties, but it is really a defense of the Galveston works and the engineers responsible for them. It was scarcely in possession of this Society before it was read in the United States House of Representatives to defeat a measure then pending, which was designed by a Committee of the House to take the direction of the improvement of Galveston Harbor out of the control of the United States Corps of Engineers. It is therefore proper to consider it in connection with the official reports of the Chief of Engineers, Gen. Newton, and the Resident Engineer, Col. Mansfield, U. S. A., which were made to Congress for the same purpose and at about the same time. The statements of the three officers together, bring into clearer prominence the combined methods by which this legislation was defeated, and enable us to form a more correct estimate of the value of Col. Merrill's contribution.

Misleading statements respecting the South Pass Jetties can now only affect the public interest by perverting the judgment of Congress in reference to similar improvements elsewhere contemplated. But those which have been made in defense of the plans of the Galveston Jetties in Colonel Merrill's paper, and in the recent official reports of General Newton and Colonel Mansfield, are calculated not only to mislead the public, but to indefinitely delay the improvement of a harbor which constitutes the natural outlet of a highly productive region, much larger in area than the entire German empire, and fully equal to that of New York, Pennsylvania, Ohio, Missouri and the two Virginias combined. The discrepancies between their recent official statements and those in their previous reports will be found most astonishing. The high positions which these officers hold, and the importance of the public works which they control, give to these contradictions unusual interest, not only to this Society and the public generally, but to every Member of Congress who desires to estimate, at its proper value, the official information which these engineers may place before him from time to time for his guidance. They are so remarkable that I have used italics in many instances, when repeating their language, to call

more particular attention to them. I have also given page and volume where the quotations I have made may be found in full, for the limits of this paper compel such abbreviations of them as may lead the reader to infer that I have unfairly curtailed them, in which case he can examine the reports in full, in the confident expectation of finding in them not only an astonishing disregard of facts, but, in the plans of Galveston Harbor, the most preposterous engineering absurdity in the annals of harbor improvement. In my address before the Senate Committee, May 21st, 1884, I exposed its incongruous features so plainly, that there seemed but one way left to save the reputation of those who are responsible for its paternity from irreparable injury, and that was to deny the facts. That these three officers have done this in a surprising manner I will show, after first commenting on Colonel Merrill's statements in relation to myself and the South Pass Jetties.

Some of his inaccuracies need explanations, which should, in fairness, have accompanied them, whilst others are altogether at variance with patent and undisputed facts. He states, for instance, that I contracted "to give a depth of 30 feet for a width of 350 feet for $7 250 000," which is an over-statement of $2 000 000. Again, he says that I earned only $1 750 000 under the original contract, and leaves his readers to infer that the remainder of $7 250 000, namely, $5 500 000, was the "large bonus" which he says "the United States was warranted in paying for them (the jetties) over the cost of construction." He includes in the cost of construction the cost of twenty years of maintenance, by which the reader is confirmed in the belief that $7 250 000 has been actually paid to me for securing the channel, independently of its maintenance. The board, which was directed by Congress, in 1874, to report the probable cost of improving the bar, estimated it at $5 342 110, and $130 000 per annum as the cost of maintaining the works and channel afterwards. I agreed to secure a channel 4 feet deeper than the board contemplated for only $5 250 000, and have agreed to maintain this channel for twenty years for $600 000 less than the estimate of the board. Therefore the statement that the Government has paid me a large bonus is wholly without foundation. Colonel Merrill states also that "the engineer who built them contracted with Congress to give a depth of 30 feet * * * and failed to secure the depth." This is equally incorrect. The depth of 30 feet was secured, and has been maintained for more than five years. He says that the obligation to produce a central depth of 30 feet added nothing to the obligation to secure a depth of 26 feet over a width of 200 feet, as a central depth at least 4 feet greater would necessarily result. This is also incorrect. The cross-section of a jetty channel depends upon the average maximum volume of water passing through it, and if the width of the 26-foot channel be increased its central depth will be decreased. This has frequently occurred in many parts of this channel. Under the law, if the 30-foot depth fails to be maintained, even by an

inch, and for a day, the pay for maintenance stops until the full depth is restored. Furthermore, under the ruling of the Attorney-General, the interest agreed to be paid by the Government on $1 000 000 security, which it holds, also stops.

Col. Merrill declares that in March, 1879, I procured the passage of an act "redistributing the payments so that I should receive the same sum ($7 250 000), for the 26-foot channel as I was to have received for the 30-foot channel." He thus not only repeats his misstatements that I get two million dollars more than I really do get, but at the same time he leads the reader to suppose that the channel is only 26 feet deep when it is 30 feet.

After making these erroneous and misleading statements, Col. Merrill says that he mentions these "facts," as he calls them, "to show the great difficulty of making results tally with predictions when dealing with large bodies of water in motion, and to show that iron-clad contracts with the Government can sometimes be set aside." This caps the climax of unfairness. He leads the reader to believe the very reverse of the truth; namely, that I had found it impossible to create so large a channel as I had agreed to, and that, regardless of the public interest, I had sought release from that obligation solely for my personal benefit. Not a word of explanation is given to save me from the odium which I would justly incur if such an inference were true, nor even to exempt the National Legislature from the imputation which is plainly implied by his statement.

The experience of Col. Merrill, and the army officers whom he defends, has unquestionably taught them "the great difficulty of making results tally with predictions," but the phrase is not applicable to the works at South Pass, so far as any predictions of mine are concerned. If the results of my works have signally failed to tally with their predictions, it is to be regretted that such inaccurate statements as these should be published about them now, for they are calculated to revive memories of a controversy of ten years ago, in which, as the world knows, the honors did not rest with those who opposed the jetties. "The large bodies of water in motion" then under discussion, have, by means as simple as the results are sublime, been so dealt with that all of my predictions are now history, and those of my opponents are resting in the grave of exploded theories, and cannot be galvanized into even a semblance of life by further discussion.

It seems passing strange that these remarkable statements should be made by one to whom the question of the modification of this "iron-clad contract" was officially referred, and who for this reason should be thoroughly familiar with its provisions.

The real facts are, that the opposition to my proposition to improve the mouth of the Mississippi by jetties, by engineers of the army, when the original bill was pending, caused the Senate Committee to insist on

the enormous channel dimensions specified, because similar dimensions had been proposed by me for the Southwest Pass, which discharges four times the volume of the South Pass, and which was the one I desired to improve, and which would have been given to me but for the opposition of the army engineers, the House having passed the bill for that Pass. In answer to my assurance to the Committee at the time, that these dimensions were not only injudicious, but dangerous, I was advised by Senators in favor of the proposal to accept them, with the assurance that when a channel such as the board of 1874 contemplated, should be secured, there would be no difficulty in convincing Congress of the impolicy of insisting upon a larger channel.

At no time has there ever been the slightest admission on my part that I could not produce the channel required. I had the whole volume of the Mississippi so harnessed at the head of the Passes in 1877, and have to-day, that it was then, and is now in my power to produce a channel 40 feet deep by 400 feet in width through the South Pass, with very little additional cost.

When the time came to increase the flow of water into the South Pass to produce the channel required, I called the attention of the Government to the danger of such a proceeding; that it would destroy the equilibrium existing between the volume of the three passes and their respective slopes of surface, and would create a tendency for the entire river, instead of one-tenth part of it (which is discharged by this Pass), to seek its way through it, because its slope is so much steeper than that of the two larger passes; and that any important enlargement of the Pass would endanger the stability of the jetties.

In my letter to the Secretary of War in 1878 (Corthell's History, page 311), I said: "We do not ask to have the depth of 30 feet lessened; but observation confirms the opinion I expressed to the committees when the bill was under consideration (in 1875), that the width required was injudicious and might involve injury to the works." At the same time I called the Secretary's attention to the fact that we had executed 80 per cent. of the works and had received only 18 per cent. of their cost, and asked that the severity of the conditions of payment should be modified, as such terms were wholly unnecessary for the security of the Government. These two questions: first, the safe dimensions of the channel; and second, my financial relief, were referred to Congress by the Secretary.

One million dollars in advance of the terms of this "iron-clad contract" was voted to me at that Session of Congress, and the fact that there was but one vote in the Senate against it, and that the vote was almost unanimous in the House, is the best possible evidence of the conviction of Congress that its terms were unnecessarily severe, and that the imputations of Col. Merrill are without foundation. The act appropriating this sum contained a provision, put in at my request, for the cre-

ation of a board of engineers to consider and advise Congress with reference to the wisdom of insisting on the specified dimensions of the channel. The act also required this board to examine and report upon the condition of the works, and also what further relief I was entitled to. Col. Merrill was a member of this board with four older members. Notwithstanding the wish of Congress to be advised as to the danger of insisting on these dimensions, the board refrained from giving any opinion on that point. It declared that "any change must be for the benefit of the contractor" (page 351, Corthell's History), and said: "In view of the various considerations presented in the report, and of the confidence expressed by Mr. Eads of his ability to secure ultimately the required channel of 30 by 350 feet, the board does not recommend any further change in the dimensions of the channel required by the contract" (Corthell, page 355).

Col. Merrill leads the reader to believe now, that I could not make "results tally with predictions," and hence sought relief from the obligation to do so, when in fact, over his own signature, with the other members of his board, he then declared that it was because "of the confidence expressed by Mr. Eads of his ability to secure ultimately the required channel of 30 by 350 feet," that they declined to recommend any alteration in the size of it!

Congress did not want to know whether I was confident or not of my ability to produce the channel. I was bound by contract to produce it, and had never intimated the least inability to do it. Congress knew that I had expressed the opinion that it was not safe to do it, because it would jeopardize the permanence of the improvement; and that I had asked that this question of insistence should be referred to a board of engineers for their unbiased opinion on this point.

Congress was not satisfied with the evasive answer of the board, and the Senate Committee charged with the matter called two of the oldest members of the board, Generals Barnard and Wright, before it, and questioned them directly as to the safety of this proceeding. They then stated that they did not think it would be safe to alter the conditions at the head of the passes; and one of them, who had been a member of the board of 1874, stated that that board had also considered this question, and was of the opinion that it would not be prudent to increase the flow into South Pass. It was, therefore, upon the testimony of two of the highest members of Col. Merrill's own corps, who at the time were members of the board to which he belonged, and also upon the concurring testimony of the board of 1874, that Congress determined, as a matter of public interest and safety, and not on my account at all, that this enormous channel should not be insisted upon.

The wisdom of Congress in not demanding this large channel, has been already shown by the fact that several feet greater depth exists below the submerged sills that were laid across the the heads of the larger passes

to control the flow into them, than before existed; and from the further fact that depths 70 and 80 feet greater than any which existed before at the head of the South Pass have occurred since the works necessary to control the flow of water into it were constructed. To have produced the dimensions originally specified would have involved the necessity of largely increasing the flow into the South Pass, whereas it has not been perceptibly increased or diminished by these works.

Col. Merrill and his board recommended that further financial relief (beyond $250 000, all of which was to be applied in the prosecution of the work) should not be given to me until after I secured a channel 26 feet deep with a width of 300 feet at the bottom, while in another paragraph they expressed the opinion that it was only possible for the jetties to create one 26 feet deep in the middle of the channel. That is, they advised that I should have no further relief until I produced what they deemed an impossible channel! And they gave this advice when $3 250-000 of the total price was still in the hands of the Government, and 90 per centum of the work was completed (Corthell, 352 and 354).

Notwithstanding this inexcusably illiberal recommendation, Congress, with great unanimity, voted me an additional payment of $750 000 without any conditions whatever, and limited my obligations with respect to the channel to "26 feet in depth, not less than 200 feet in width at the bottom, and having through it a central depth of 30 feet."

The report of this board will be found rich in expressions of approval of the work, and of the certainty of its success, to the extent of possibly 25 or 26 feet central depth. The secret of its profuse approval will probably be better understood by reference to the report (page 365 of Corthell's History), in which it assumes that I was simply carrying out the plans of the board of 1874, and says: "No reason is known why Congress should not have left the execution of the work to its own agents, except that the present contractor, Mr. James B. Eads, offered to accomplish the results contemplated by the board without payment, unless those results are secured." In other words, Col. Merrill's board proposed, very innocently, to deprive me of whatever credit I was entitled to for originality in my plans and the success which attended them, thus lessening the equity of my claim upon Congress for the relief which I sought.

In reviewing the report, I referred to this statement and its unfair conclusions, and in self-defense stated that I urged the jetty plan of improvement as the only proper one, early in 1873, and that in the winter following I made a formal proposition to Congress to deepen the mouth of Southwest Pass by Jetties, and offered to guarantee their complete success before any member of the Corps of Engineers had ever expressed, officially or publicly, any preference whatever for this method, and before the Board of 1874 was even thought of; and, fur-

thermore, that the plans I was executing were my own and not those of that board.

Congress was guided by the report and estimate of the board of 1874 in making this contract, and it has secured by it a channel fully one-quarter larger than that board contemplated, and for $92 110 less than its estimate. It has secured, also, the maintenance of this channel for twenty years for $600 000 less than the price estimated for the maintenance of a smaller one, and it holds $1 000 000 security for the remaining fourteen years of maintenance. Yet, with all this, Colonel Merrill is not satisfied, but leads his readers to believe that the Government has been overreached by me. It was Congress, and not I, who selected this little pass, and if, after careful investigation, it restricted the size of the jetty channel to the limits of safety, and thus disregarded his advice, that is no reason why any one should misstate facts or feel hurt about it six years afterwards. It should be sufficient consolation to him to know that the Government has secured the largest channel which it is possible for the normal discharge of this little pass to safely maintain, and one which is four or five feet deeper than he ever thought possible.

The Fort St. Philip Canal.

The Government, in 1873, again imposed upon some of its army engineers the question of opening the mouth of the Mississippi, a problem with which they had frequently wrestled in the previous forty years. In 1874 a board of seven officers recommended a canal from the river near Fort Saint Philip to deep water in the Gulf. Its estimated cost was $13 000 000. Its actual cost would, doubtless, have reached $20 000 000, and at the usual rate of Government works it would have taken twenty years to complete it. Not one of the entire corps opposed this ill-advised scheme, excepting General Barnard, certainly the ablest and most accomplished civil engineer in the army.*

In 1874 the House had actually voted $8 000 000 to begin this canal with, when the measure was defeated by the jetty bill. In one-fourth of the time, and with one-fourth of the money required for the canal, an open channel to the sea, through the jetties, in every way better and deeper, was secured for the vast empire drained by the Mississippi River. Not a single ship in the past six years has been detained one moment for lack of deep water through this channel. Before we com-

* General Barnard, in his minority report on the Fort Saint Philip Canal, dated 29th January, 1874, discusses, first, the method of dredging or stirring up the bottom, and, second, that of jetties. With respect to dredging, he says: "By reference to the best authority, I have proved the adequacy of dredging operations on the bar by well tested means; but I think there is yet room for improvement, and especially in diminishing cost." He further says of dredging, that the attainment of 20 feet depth on the bar has by no means been established to be the maximum, and adds; "As to that depth, however, we have the strongest assurances." He discusses the jetty method of improvement with great ability, and says: "The question submitted, however, is not so much to recommend its trial (the jetty system) as to recommend its consideration, and that scrutiny and survey on which alone estimates can be based."

menced the work, thirty-five vessels in one day alone were reported to be waiting inside of the bar to get out to sea; three were aground on the bar; and seventeen were outside waiting to come into the river. These facts constitute an insuperable bar to all adverse criticism from the Engineer Corps of the United States Army. Statisticians have estimated the annual saving in the cost of transportation which has resulted directly from the jetties, and from their influence upon other routes to the seaboard, as high as $100 000 000 per annum. Assuming that this saving has only been one-tenth as much, the country and the Engineer Corps of the Army are still vastly better off than they would have been if we had not built the jetties. The splendid results achieved by them, their low cost and rapid construction, compared with the canal, all conclusively prove now, that the canal was not the proper solution of the problem.

Galveston Harbor.*

The harbor works which have been in progress during the last ten years at Galveston, having produced no appreciable increase of depth on the bar, the representative men of that city addressed a request to me in 1883 to undertake to secure the requisite depth on terms similar to those under which the works at South Pass were executed. This I consented to do, on condition that they would secure the necessary legislation. Accordingly a Bill (No. 1652) was introduced in the Senate, authorizing me to obtain a channel 30 feet deep at a maximum cost of $7 750 000 (the works required being two and a half times greater than those at South Pass), and payments being conditioned wholly upon securing stipulated depths. This proposal was indorsed by the entire Congressional Delegation of Texas; by the leading citizens of the port; by its Mayor and Council; by the Legislature of Texas; and by almost the entire press throughout the immense territory of which Galveston is the chief seaport.

Because of the misrepresentations with which my offer to deepen the mouth of the Mississippi in 1874 had been assailed, and the personal annoyance incident thereto, I declined to appear, even before the two Committees at Washington, to advocate my proposal, although desired to do so by the unanimous request of the Congressional Delegation from Texas. On my return, however, from England, I read with astonishment the very unfair reports of Gen. Newton and Col. Mansfield upon it, and believing that if their misstatements were not corrected I would rest under the imputation of having fixed up a scheme to extort a large sum from the public treasury, I asked to be heard by both committees in its defense; and I particularly requested that Gen. Newton should be invited to be present. I then explained the plan of these

* The maps which are herewith appended and referred to in the text, are reduced by photo-lithography from the original ones in the official reports referred to, excepting the comparative Plate No. XIII, which was carefully prepared by Mr. Corthell from the other four.

officers, showing conclusively that it was never designed to produce more than 18 feet, and that, owing to the great width of the jetties, the huge gaps in them, and their low heights and short lengths, *no improvement whatever could be expected from their completion.* My argument was so unanswerable that they had no alternative left but to deny that their plan embraced these fatal defects, or to admit the conclusion.

The Senate Committee on Commerce, after hearing my arguments in favor of, and General Newton's against the Bill, reported it favorably to the Senate. The River and Harbor Committee of the House had previously placed on their Bill $250 000, with which to continue the work of the army engineers. But the Texas delegation unanimously authorized a motion that this sum be stricken from the Bill, and this remarkable motion was made in the House and adopted. This delegation then urged that Senate Bill 1652, embodying my proposition, should be made a part of the River and Harbor Bill. The Committee objected to this, and as a compromise, reported a much larger appropriation for Galveston Harbor, and *without my knowledge or request*, put a clause in the Bill providing that I should be placed in charge of the works upon a salary. The Bill containing this, and probably one hundred other appropriations, was finally defeated.

The Jetty System Explained.

It is known to all engineers familiar with harbor improvements, that the jetty system is a method of deepening and maintaining a channel across a shoal by such works as shall compel the water flowing over the shoal to pass through a narrower channel; but, as this paper will be read by some who are not familiar with harbor improvements, I will be pardoned for stating the following elementary principles which control such problems :

First.—A current is caused by the fall of water from a higher to a lower level, which fall is indicated by the slope or inclination of the surface of the water.

Second.—Friction of the bed over which the water flows is the chief force opposed to the current.

Third.—The velocity of the current will be increased by either increasing the slope of surface or by increasing the volume, or by lessening the friction.

Fourth.—The friction increases as the width of the bed increases; that is to say, if the bed of the channel be twice as wide, the friction will be doubled; if its width be reduced one-half, the friction will be reduced one-half.

Fifth.—The power of water to transport sand increases with the square of the velocity.

As every grain of sand that lies upon the shoal is brought by the cur-

rent and left there, because it could take it no further, it follows that we cannot hope to deepen the channel, except by increasing the current. This can be done in three ways, but in this case it is impossible to do it *at first*, except by increasing the slope of surface. After the current is *increased*, deepening commences, and as the contracted channel enlarges, the slope is gradually reduced to its former inclination. When this occurs the old rate of current would return, but the other two elements, namely, *increase of volume* and *diminished friction*, then continue the abnormal acceleration. As the deepening goes on a *reduction of slope* follows, and this finally reduces the velocity so much that deepening ceases. The slope results from the difference of the heights of the water in the Bay and the Gulf. *To facilitate* the flow from one to the other, by *leaving openings through the jetties*, tends to keep these heights nearer alike and the resulting slope between them must then *be less*, not greater. This is precisely what we do not want *at first*.

With these simple principles before us, the reader will be amazed at the fact that the engineers at Galveston actually proposed to leave two enormous gaps, each more than a mile wide, between the land and the inner end of each jetty, *to facilitate the inflow into the bay!* The frictional resistance of the bed of the channel was absolutely ignored in planning the works, for they placed their jetties 2¼ miles apart. In addition to these errors, they proposed to build up only a small portion of their isolated works to the level of *mean low tide*, overlooking the undermining effect of currents flowing over their tops, and as an escapement of the ebb tide would be made, not only through the enormous gaps, but over the whole line of the jetties, no increase of current could possibly occur to deepen the channel. They furthermore limited the length of one jetty to the 12-foot contour, and the other one was to extend only to the depth of 13½ feet; and with these they expected *to maintain* an 18-foot channel, overlooking the fact that the waves of the sea would fill up any deeper channel beyond the jetties temporarily caused by the outflow than the depth in which the jetties terminated. The waves constantly transport the sands on the bottom towards the shore, not from it; therefore, the jetties must extend out to a depth where wave action is not felt. This depth depends on the maximum height of the waves. In the Gulf, wave action extends to about 25 or 30 feet depth. If the jetties do not extend to this depth, shoals will inevitably be thrown up by storms in front of the jetties.

Let us now confront the statements of these engineers, made since my exposure of these several radical errors, with their previous official records.

Width Between the Jetties.

It is so evident *now* that the great width between the jetties is a fatal error in their plan, that they have squarely sought to escape this

dilemma by declaring that the North Jetty was never located, and that the width between them has therefore never been determined.

Col. Merrill says the statement "that there is to be two and a quarter miles between the jetties is positively denied by the Chief of Engineers and by the resident engineer, both of whom state that the North Jetty has not yet been located."

Major Howell, in 1873, designed the plan which has been in process of construction at Galveston. In 1874, it was submitted to an advisory board, composed of General Newton (the present Chief of Engineers) and two older officers. Major Howell, in his first report (Chief of Engineers' Report of 1874, page 732) says: "it is proposed to construct *parallel jetties* along the lines marked C D and E F on the chart * * * they may be called *submerged jetties*, since they will not, except *on a small portion of their lines*, be built up to the plane of mean low tide, while *for the greater part their tops will be five or six feet below that plane* * * * they are calculated to give a depth on the outer bar of between *eighteen and nineteen feet.*"

Herewith I submit the chart (Plate No. IX) accompanying the report of Major Mansfield for 1880, on which is placed, *over his own signature*, the words, "showing lines of proposed jetties." On this chart, the north line has the word "gabionade" on it, and the letters D E F. This is the line of the North Jetty, which, it is claimed, has never been located. Major Howell refers to the jetties as "parallel jetties," and by applying the scale of this chart it will be found these jetty lines are over twelve thousand feet (or 2¼ miles) distant from each other. On page 1225 of this same volume, is this statement: "Captain Davis' reports showed that from its commencement, April 14th, 1877, to January 1st, 1880, the Bolivar Pier (the North Jetty) had been extended seaward from the beach 10 220 feet." Nearly two miles. And on page 1228 of the same volume we find the following: "On the North Pier 7 000 linear feet (of gabions) were put down, apparently on the assumption that the success of the system had been established at Fort Point." As both of these statements were signed by General Newton, in 1880, *before* I had exposed the inexcusable blunder of placing the jetties 2¼ miles apart, it is evident that his *recent* statement and those of his subordinate officer are inaccurate. The North Jetty was not only located, but nearly two miles of it were actually constructed prior to January 1st, 1880. At page 1229, Appendix N, Chief of Engineers' Report for 1880, will be found a statement, signed by General Newton, as follows: "From the direction taken by the cylinder channel, it would appear that the direction of the *North Jetty, for a distance of two miles from the shore, need not be changed.*" This is the jetty we are *now* told was never located!

An additional proof that the jetties were both located, and that they were to be two miles and a quarter apart, will be found on page 1213,

Chief of Engineers' Report for 1880, where Mr. H. C. Ripley, the civil engineer assisting Col. Mansfield, says: "There is maintained on the bar, over the area between the two jetties, a very constant depth of eleven feet, with area of discharge of 140 690 square feet."

How did Mr. Ripley know that the area of discharge between the two jetties was 140 690 square feet, if either jetty was not located?

This area of 140 690 square feet divided by "a very constant depth of eleven feet," gives 12 790 feet as the width between the jetties. Col. Mansfield, on page 1206 of this same Report, says: "I invite attention to the accompanying report of Assistant Engineer H. C. Ripley, to whom I am greatly indebted for so clear and comprehensive an exposition of *our plans and estimates* for the improvement of this harbor."

In the report of General Newton (January 17th, 1885) to the River and Harbor Committee, he says: "It is proper, likewise, to say that the board (1874) never indorsed the system of parallel jetties, 12 000 feet apart, devised by Major Howell, as competent to afford even a draught of 18 feet." This statement is in conflict with the following previous ones made by this officer:

In 1874 (see page 1267, Report of 1880), the board said: "That, if piers proposed by Capt. Howell were constructed, extending over the bar, the depth of water thereon would be increased in an important degree, though exactly what depth might be looked for cannot be predicted."

On the next page, this advisory board recites what it said in 1876, when it was again convened. It is as follows: "That the two jetties or training walls, *recommended* in the report of January, 1874, will, if constructed, *produce an important increase of depth over the outer bar.*"

In 1874, Gen. Newton signed this statement also: "The two piers will therefore have the effect, in the opinion of the board, not only of improving the outer bar, but also, incidentally, the inner bar at the entrance to Galveston Harbor." (See Report of Chief of Engineers, page 737, 1874.) Here we see that Gen. Newton and the board *did indorse* "the system of parallel jetties, 12 000 feet apart, devised by Major Howell." They not only indorsed them, but advised the building of Major Howell's gabions in these words: "By first constructing the inner end of the pier on the Fort Point side, and if found to succeed there, the pier on the other side be commenced at its shore end." Major Howell's plans were then before the board, and the location of these two piers was indicated by the lines *C D* and *E F* on his chart. (See same Report, page 738.)

From these stubborn facts it appears that the location of both jetties at this distance from each other was *approved*, and the construction of each *was authorized* over the signature of Gen. Newton; and furthermore, that the location of both was *certified to at this distance apart by Col. Mansfield*, in 1880, on his chart (Plate No. IX).

Should some charitable apologist say that these gentlemen only meant that "the North Jetty has not been located" *since the failure of the first attempt to build it*, even this poor excuse would be debarred by their own official testimony. After assuring the Senate in the spring of 1884, that this jetty had never been located, the Chief of Engineers in his Annual Report, 1884, page 216, says that, "*under the project* of 1880, $980 662.51 was applied to jetty construction, viz.: 90 *feet on the north side* and 22 551 feet on the south side of the entrance prolonged Gulfwards." He thus not only contradicts his statement to the Senate in the spring, but gives another proof that this North Jetty *was located under the modified project of* 1880, and that a part of this gross sum was expended on its construction. Under the heading of "*The Official Plan*," on another page will be found quotations from page 1059, Chief of Engineers' Report, 1883. From them it will be seen that in June 1880, Col. Mansfield actually commenced the initial works of the second attempt to build this jetty, on the line of Major Howell's gabionade, and that *he reported it, as late as* 1884, under the head of the "*North Jetty*," to be in good condition, and located "at the outer end of the Bolivar gabionade." (See page 1295, Rep. 1884.) On the chart accompanying his report (see Plate No. X) will be found a dotted line, showing the original location of the North Jetty, or Bolivar gabionade, made by Major Howell, and at the other end of it, 10 000 feet from shore, a little square figure marks the spot where the new method of construction of this jetty was commenced by Col. Mansfield in June, 1880. This chart is signed by Col. Mansfield.

Six months after this report of 1883 was written, I said in reply to the invitation of the people of Galveston, December 6th, 1883: "When I visited the Maas in 1874, I thought Mr. Calland had placed his jetties much too wide apart (1 000 meters). I think this was the chief cause of his failure. This mistake (as I think it was) has been made at Galveston and to a much greater extent" (about 3 700 meters). When the reports of Gen. Newton and Col. Mansfield were made to the Senate, three months after I wrote this, Congress was then assured, for the first time in ten years, that "the North Jetty had not yet been located!" Comment is unnecessary.

Outlets Through the Jetties.

As the reader may not at once perceive the mistake of leaving these openings to facilitate the flow into the bay, I will explain that, in a state of nature, the immense frictional surface over which the flood water has to pass in filling the bay retards it so much, that the tidal rise in the Bay at Galveston is probably 10 per cent. less than it is outside of the bar. On the other hand, the discharge of the ebb is retarded by the same cause, making the total rise and fall just inside of the harbor probably 20 per cent. less than it is at the same depth outside of the bar.

The bottom of the channel is composed of separate grains of sand, without the slightest cohesion or bond. They were brought here by currents and left, because those currents, from some cause or other, became too feeble to carry them further. Now, the reader must remember, first, that a current depends upon *the slope of surface* of the water* (not upon the slope of the bottom), and second, that the excavating or transporting power of the water depends upon the *velocity of the current.* This excavating power or *velocity of the current must be made greater* than it now is over the bar, or the channel cannot be permanently deepened by any artificial structures known to the science of engineering. And yet no engineer can increase this velocity *permanently*, as will soon appear. But the velocity *must* nevertheless be increased, or these masses of sand composing the bar cannot be *permanently* removed. How can the velocity of a current be increased? In three ways. First, by increasing the *slope of surface*, which is the cause (or rather the exemplar of the cause) of the current. Second, by lessening the frictional resistance of the bed. Third, by increasing the volume or mass of the water. We cannot avail of these last two methods *at first*, but after we shall have *once increased* the velocity by permanent works, these two last means will then ultimately come to our aid. *At first* we have absolutely no power to increase the velocity of current but by *increasing the slope of surface.* These slopes are alternately towards the sea and from the sea, accordingly as the tide fills and empties the bay. They are by no means steep, as the average maximum tides are but a little over a foot high. The force of the present currents resulting from these low slopes cannot overcome the force of gravity in the grains of sand composing the bed of the channel. With the present slopes, volume and frictional resistance, the equilibrium between the *velocity* or transporting force of the current and the force of gravity in the sand is reached when the depth of 13 feet occurs on the bar. If we should suddenly deepen the channel by dredging, it would be larger than nature demands under the present conditions. The bay would then fill more easily, the slopes would be lower, the velocity less, and soon after the dredging ceased, the enfeebled current would drop its burden of sand in the enlargement, and restore the present depth. On the other hand, if we could suddenly drop a million tons of sand in the channel, and reduce the depth a few feet, what would occur? The bay would not fill so rapidly, and steeper slopes would occur in consequence. These would create greater current velocities, and the removal of the obstruction would be effected in a little while; and, with the recovery of the 13 feet, the bay would fill more

* At New Orleans the surface, in flood, is 14 feet higher than it is at the head of the passes (97 miles below). At the former place it is 150 feet deep; at the latter it is only 30 feet. Therefore the bottom of the river slopes upward to the sea in this distance. If the river bottom at New Orleans were raised so as to give it the same regular downward slope to the sea which the surface of the river has, the current would be much slower. The river would have that much less volume (or mass), and volume is one of the elements producing the current.

easily, the slopes would diminish, and the former velocities of current would be restored. The slope results from the difference of level between the surfaces of the Gulf and the Bay. Any one can see that the more freely the water can flow from one to the other, the less difference will there be between the heights of their respective surfaces; and, consequently, the less will be the slope of surface through the channel between them, and the less will be the current velocity. This is certainly the very opposite of what *we must have* to produce deepening. We must have more rapid currents, and nothing but steeper slopes will produce them. Hence it is evident that *large openings*, and a freer communication with the bay through them, and over low, submerged jetties, are precisely what we do not want.

Let us suppose that jetties were, at mean tide, suddenly placed across the bar, 3 000 feet apart, and extended from 30 feet depth inside to 30 feet depth outside, and were so high and tight that all the water which passes in or out of the bay is compelled to pass through them. When the tide rises it will be impossible for the bay to fill as fast as before. The tide outside rising only to its normal height will produce a steeper slope through the jetty channel than existed in that locality before, because the surface of the bay must be lower, as it will have been deprived of a large part of its normal volume of the flood. This steeper slope will produce a more rapid current into the bay, over sands which are only quiescent under the present velocity. If this slope increases that velocity from 3 to 4 miles per hour, which is not an unreasonable supposition, the transporting power of the water will be increased from 9 units to 16 units. Wherever through this jetty channel the depth is least, there will the current be most rapid. Hence the inflow through it will have the effect of cutting down the high parts of the bottom first, and transporting these sands into the deeper places. In a few hours this whole process is reversed, and a steeper slope than before will exist in the opposite direction, and the sands of this contracted channel will be swept out to the sea, and will there be distributed by littoral currents, or sea waves, out of the way of the outflow from the jetties. Of course, the greatest increase of slope will only occur at first, and at the highest and lowest periods of the tide; but *as the channel deepens the volume increases, and the ratio of friction to volume decreases*, hence lower slopes will then produce equally rapid currents. Therefore the bay will ultimately fill more and more quickly, and these abnormal currents will be thus toned down until the deepening ceases and a new condition of stable equilibrium, or what engineers call "a new regimen," is established between the maximum velocity of the current and the force of gravity in the sand. When this is reached it will be found that the tidal movement in the harbor conforms more nearly to the rise and fall in the Gulf, both in height and time, and that the slopes are *lower* than at present. Because the deepening of

the channel will continue until the bay will again receive all of the water due to the difference in the tidal movement, and to the normal slopes of surface occurring between the alternating levels of bay and sea through this deep channel, and as the friction which now retards the entry and exit of the water will be greatly reduced through it, a correspondingly increased quantity of water will enter the bay.

Any excessive increase of tide caused by winds will have the effect of permanently increasing the depth of this jetty channel, and of making it more capable of discharging such excessive quantities of water again. By such abnormal causes the channel will be so deepened that the currents from ordinary tides will not be sufficient to disturb its bottom. The greatest work of excavation is done when the velocities are greatest. The channel with high jetties extending out to deep water would get no accessions of sand, and the depth would be permanent. It must be apparent, from this explanation, that the object sought by these officers, namely, *facilitating the inflow of the tide* into the Bays at Galveston, Charleston, Sabine Pass, etc., by leaving openings through the jetties near the shore, is precisely the way *not to do* what they want to accomplish. Besides losing through them the most important agent for deepening the channel--the ebb tide—they will admit into the channel all of the sand which the sea waves can bring. *Submerged jetties* simply aggravate the evils caused by these openings. They really extend the openings over the entire length of the jetty. They prevent, by increased friction, the rapid inflow to fill the bay ; they permit the loss of the ebb tide, and thus lessen the force that should be applied to deepen the channel; and they allow the waves to transport sand over them into the channel. Openings and submerged jetties are therefore an inexcusable violation of the very essence of the jetty system.

Of course, the width between the jetties must be determined by the length of the channel and the slope of surface which can be had through it. The ratio of friction to volume increases as the volume decreases ; hence by making the jetties too narrow, we may so decrease the volume flowing between them as to neutralize the benefit we would otherwise gain by the temporary increase of slope, which is at first absolutely necessary to induce the deepening. To place them too close together would be as fatal as to place them too wide apart; or, as to leave the proposed openings through them. Within certain widths the resulting channel will be deeper in proportion as the jetties are nearer together, and here again comes into the problem another important matter—that of maintaining the works—for if the channel be too deep, the jetties may cave in or settle. Their direction must be determined with due regard to the prevailing winds and sea currents, as well as to the depths in which they must be built. For these reasons the location of the South Jetty is an exceedingly injudicious one.

After I had shown the fatal mistake of leaving the enormous outlets

between the jetties and the shore, these gentlemen positively denied that such gaps ever existed in the plan. Col. Merrill says: "I can find "no authority for the existence of these outlets; nothing of the kind "appears in the last official map of Galveston Harbor."

The first proof on this question will be found in the Report of Chief of Engineers, 1874, where, on page 733, we are told that the jetties "will not, except on a short portion of their lines, be built up to the plane of mean low tide, while, for the greater part, *their tops will be five or six feet below that plane.*"

We also find on page 1212, Report of 1880, this statement:

"About 2 000 feet from its inner end the first concentration of current against the jetty occurs; it would seem appropriate to raise the jetty to its full height, but, inasmuch as a *larger opening* must be left for the admission of flood water into the bay, and the inner end seems to be the only available place *for an opening*, aside from the opening between the jetties, this portion of the jetty is raised only sufficiently to destroy the scouring force of the current. * * * In the case of the North Jetty, it will be seen that *the opening* between its inner end and the shore is covered by a shoal extending from Bolivar Point."

Again, on Col. Mansfield's second chart (Plate No. XI, herewith submitted) accompanying his Report for 1880, a vertical longitudinal section of each jetty is shown. It will be seen that the inshore end of the North Jetty begins at letter E, and rises through 2 000 feet to the level of low tide, leaving a gap 8 000 feet long over the bottom between the shore end of it and the beach. On page 1213, Mr. Ripley estimates the "area of discharge *opening at inner end of North Jetty at* 55 370 square feet." This ought to be a big enough gap to have attracted the attention of any one industriously hunting for it. No estimate of the cost of closing this 8 000 feet gap is to be found in any of the reports.

The gap in the South Jetty is shown on the chart (Plate No. XI) to be more than a mile long; and from this gap seaward, for a distance of about 10 000 feet from the shore, the jetty rises only to five feet below the level of mean low tide, thus making the entire gap between this jetty and the shore about 10 000 feet long, and 5 or 6 feet deep at low tide.

Again, on page 1212 (Report of 1880) will be found the following:

"In the adoption of the form of profiles for the jetties we have been guided by the principle that, while the ebb current should be confined to a single channel across the bar, as little obstruction as possible should be placed to the admission of water into the bay during flood tide. * * * *Hence the inner ends of the jetties commence some distance from the shore.*"

On page 1414 of the Report of 1882, is a statement, *over General Newton's signature*, as follows: "It seems probable that they (the jetties) must be connected with the shore." Why should they be connected with it, if it had not been proposed to leave these gaps between them and the shore?

These proofs could be largely increased by further quotations from

the Reports, but enough have been given, I think, to cause the reader to peruse with astonishment the following extract from General Newton's Report to the River and Harbor Committee (January 17th, 1885):

"I do not perceive the relevancy of Mr. Eads' remarks concerning leaving outlets, which really have no relation to what has been done or what is intended to be done here. He has, probably, seen a progress sketch of the South Jetty at Galveston up to the date of June 30th, 1882, or some unfinished plan not authorized for construction, and has been swift to conclude and to publish to the world that *outlets* were to be left in the jetties near the shore."

After I demonstrated the fact to the two committees that these openings are in direct violation of the principles upon which the jetty system is based, we find the highest officer in the Engineer Corps of the Army "swift to conclude and to publish to the world," in the face of these statements, this positive denial that he and his brother officers ever countenanced such a ridiculous and preposterous proposition as that outlets were to be left in the jetties at Galveston, while for eight years after these works were begun in 1874, *low jetties* with *shore openings* were approved by them at several other harbors. On page 559, Chief of Engineers' Report, 1878, in discussing the plan of the jetties at Charleston, S. C., we learn that the conditions which have been kept in view in preparing the design have been:

"*First.—They should not impede the flow to such a degree as to prevent the tidal basin being filled, as now, at every influx of the tidal wave.* To this end, the inner half of each jetty, more especially its central portion, located in deep water across the thread of the current, *is kept several feet below the water.*" Just above this we read: "The inner portions, crossing the beach and main channels respectively, *might, perhaps, be kept low enough to allow the smaller classes of vessels to pass over them.*"

On page 572 of this Report it will be seen that General Newton was a member of the advisory board that reported favorably upon these jetties.

The opening through the South Jetty at Galveston was found (as should have been expected) to admit large quantities of sand, which constituted what these engineers call the *inner bar.* This opening embraced what was known as the "swash channel." The City of Galveston partially closed this with a pile dike, extending from the shore about one mile out, by which means the depth on the inner bar was largely increased. The South Jetty is an extension of this city dike. It was to prevent injury to the inner bar that the opening designed to be left in this South Jetty in 1880 was closed in 1883. The advisory board was only then, after eight years of study and observation, beginning to see the folly of leaving openings "to facilitate *the inflow of the tide.*" They admitted large quantities of sand. Hence we find the advisory board, in 1882, saying: "It seems probable that the jetties must be connected with the shore." Another evidence that they began about

this time to get a little of what General Newton (in his Report to the Senate on my proposition) calls "a much needed light" on the jetty system, is to be found over his signature on page 1439, Chief Engineer's Report, 1882, in relation to the Jetties at Sabine Pass. It is as follows:

"It is an objection *to these openings*, as well as to *the submerged jetties*, that they will permit material to be brought into the pass."

The Official Plan and Estimate.

The following extracts from the Reports of these officers to the Two Committees in 1884–85, assure Congress that with $750 000 and two years' time "the official plan" can be completed, and what I proposed can be obtained with one-tenth of my offer. To know how near the truth is approximated in this assurance, it is necessary to know: *First*, what is the *official plan; second*, what is the *estimate* of its cost; and *third*, if completed, will it produce the depth I proposed.

In his report to the Senate, March, 1884, Colonel Mansfield says:

"*The South Jetty has been completed* for the sum already appropriated; the plant is now on hand, and we have the experience that will certainly enable us *to complete the North Jetty within the estimate, and still leave a margin for dredging, to facilitate the scour*, if it should be found necessary or desirable to hasten it; and not over two seasons will be required for the expenditure of this money ($750 000), and it is reasonable to expect by that time will be solved the vexed question of deep water on Galveston Bar.

"Under the present system of work, *adopted at the close of the year* 1880, the estimates for the entire completion of the two jetties for confining the outflow of Galveston Bay, *looking to obtaining 25 feet of water*, were: for the South Jetty, $1 080 448; for the North Jetty, $658 525; for plant, $40 000; contingencies, $46 480." (See *A*, Plate XIII.)

Gen. Newton, in his report to the Senate on my proposal, says (March 27th, 1884):

"The North and South Jetties, placed according *to the official plan*, can by further extension into deep water, and by the construction of auxiliary works if needed, be made to develop all the depth of channel which the nature of the locality will admit."

As the last meeting of the advisory board occurred in 1882, it is evident that "the present system" referred to above by Col. Mansfield is "the official plan." Now, turning to his Report, page 1059, for 1883, we find the following:

"On 30th June, 1882, the work for the improvement of this entrance *under the approved project* of 1880 was in good condition. It consisted of

"*First.*—North Jetty: A 'trial section' of mattress work with concrete ballast, 90 feet long, 60 feet wide, and 2½ feet high above the sand bottom, *placed in June*, 1880, at the *outer end of the Bolivar gabionade.*"

"*Second.*—South Jetty: Mattress work with stone ballast, placed in 1880–82, extending from pile breakwater, etc. * * * *."

This statement, though somewhat grandiloquent when the magnitude of the North Jetty works are considered, is important as proving—1st.

That these are the identical works contemplated by "the approved project of 1880." 2d. That the North Jetty was then located, notwithstanding Gen. Newton and two colonels of the Engineer Corps had assured Congress that it never had been. 3d. That its location was fixed at the outer end of the buried remains of the old gabionade. Therefore, this North Jetty of "*the official plan*" of Gen. Newton, was actually upon the same line upon which Major Howell, in 1873, located his gabionade *two and a quarter miles distant from the south one*, and which was, as I have shown on page 295, approved in 1874 by Gen. Newton; and 4th. This quotation shows that the "official plan" or "approved project of 1880," was in full force up to July, 1883. The *outlets* through the jetties; the *great width between them*; the shallow depths of 12 and 13½ feet respectively at which their sea ends terminated; and the grand escapement of the ebb tide over the whole lengths of the jetties, had up to that time given these gentlemen no anxiety whatever. Nine months later, after I had pointed out these four fatal defects, either one of which is sufficient to prevent absolutely the securing of even 18 feet of depth, they fully recognized them, and forthwith flatly deny the outlets and the excessive width; extend their unlocated North Jetty out to deep water, and assure Congress that their plan contemplates raising the works above high water also. This is, of course, all very complimentary to me, but it is as inexcusably deceptive to Congress and the public.

Col. Mansfield says:

"The project contemplates the raising of this work to a greater height if necessary and the construction of a jetty on the north side, *the position of which is yet to be determined.*"

This statement, read to the House last January, is squarely contradicted by his own Report of 1883, which shows that the position of the North Jetty had been determined in 1880, and that the second attempt to construct it had been commenced by this very officer in that year, on the identical line which Major Howell selected in 1873. Furthermore (see page 294), the advisory board that approved this location in 1874, said in 1880 that the direction of the North Jetty "need not be changed." Nor did the project contemplate raising the works to a greater height than low tide, nor even to that height, but only to an average of 5 feet above sand bottom. The Advisory Report of 1880 (page 1267) expressly says that the jetties are to be "submerged," and that they are "to be extended to such distances as would, by contracting the waterway, deepen the Gulf Bar to a possible draught of 18 feet.

Cost of the North Jetty.*

Having clearly identified "the approved project of 1880," and the

* After a careful perusal of the three following paragraphs, the reader should examine Plate No. XIII, on which the vertical section of the North Jetty, according to the "approved project," will be found, marked *A*, and the location and length of it at *B*, extending from *E* to *F*. These are copied from the chart accompanying Col. Manfield's report of 1880, in

estimate of its cost, we are prepared to analyze the methods used to convince Congress that "about all that I proposed" can be done for $750 000. The quotations I have made from Col. Mansfield's Report under the previous heading, contain an unequivocal assurance in March, 1884, that the plan he was then executing was adopted in 1880. That the South Jetty *was* then *completed* for the estimate, and that "we have the experience that will certainly enable us to complete the North Jetty within the estimate," and still leave a margin for dredging. And finally, that the estimate for this North Jetty is only $658 525. In this Report this jetty is described as being 14 340 feet long, and terminating in 20 feet depth. These are all positive assurances made in 1884; but at page 1212 of the Report of 1880, the North Jetty is stated to be *only* 8 090 *feet long*, and on page 1217, where the estimate of its cost is given at $658 525, its length is again stated at 8 090 feet. And instead of its extending out to 20-feet water, we find on page 1212 that it is to stop at 12 feet. Consequently Col. Mansfield has added 6 250 feet to the length of this jetty, all of which is to be built in deeper water than the portion which was estimated to cost only $658 525, and he has done this, not only *without the least intimation* that this mile and a quarter *is not covered by the estimate of* 1880, but with the positive assurance that it can be completed within that estimate, "and still leave a margin for dredging." On page 1212 of the Report of 1880, we are further told that the inner end of this jetty commences "near the outer end of the Bolivar gabionade." This gabionade, or North Jetty, was built out 10 220 feet from the shore (see page 1225, Report of 1880), but at that time it had been engulfed in the sands. Consequently Col. Mansfield and Gen. Newton must have known that a huge gap between the shore and the inner end of their proposed North Jetty existed in the plan of 1880, *the closure of which was not then contemplated.* The Charts of the Report of 1880 show this gap to be 8 000 feet long, and on page 1213 of that Report, the computed area of this opening is stated to be 55 370 square feet, or about twice the size of the South Pass.

At this late date every one at all familiar with the jetty system knows that a jetty terminating in 12 feet of water cannot maintain 25 feet depth. Hence it was necessary to carry it out a mile and a quarter further to 20 feet water, or abandon the hope of convincing Congress that such a jetty would give the depth of 25 feet. Again, it was made perfectly clear to these committees by me, that the outlets proposed would be absolutely fatal to their success, hence it was necessary to deny that any were intended in "the official plan."

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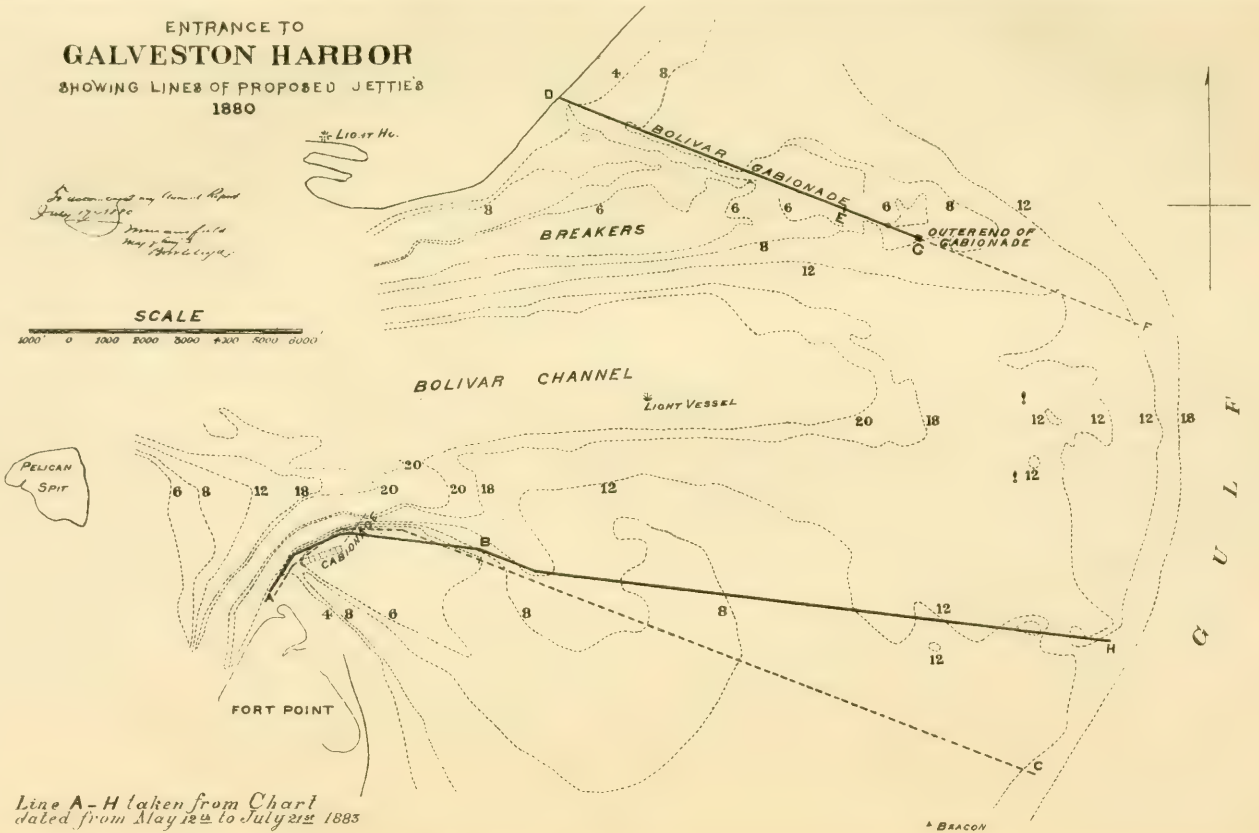
which the length of this jetty is stated to be 8 090 feet, and its cost $658 525 (see Plates IX and XI). The jetty described in March, 1884, by Col. Mansfield in his report to the Senate Committee, and submitted *with this estimate*, is shown at *C*, and is 14 340 feet long. It will be seen that its location is in much deeper water, being much nearer to the Bolivar Channel. It is copied from his chart (Plate No. XII) in his report of July, 1884. The gap or outlet between the North Jetty and the shore is shown between *D* and *E* on the sectional elevation *A*, and is 8 000 feet long.


ENTRANCE TO
GALVESTON HARBOR
SHOWING LINES OF PROPOSED JETTIES
1880
Lig.st Ho.
SCALE
1000' 0 1000 2000 3000 4000 5000 6000'
BREAKERS
BOLIVAR CHANNEL
Light Vessel
PELICAN SPIT
FORT POINT
BOLIVAR GABIONADE
OUTER END OF GABIONADE
CABIONADE
Line A-H taken from Chart
dated from May 12th to July 21st 1883
Beacon
GULF

The annals of engineering may, I think, be challenged in vain to find so grave and stupendous a misconception as that which was caused in the United States Congress by the statements of these officers. It was assured that the South Jetty was completed for its estimate when it was not completed, and that $750 000 was ample to complete the north one. That the estimate of the cost of this one was only $658 525, and it was described by the officer who made this statement, and almost in the same breath, as a jetty starting from a point 8 000 feet from shore, and extending thence 14 340 feet seaward, and terminating in 20 feet of water, while he knew that the estimate submitted with it was made under his own supervision in 1880, for a jetty only 8 090 feet long, no part of which was to be in water over 12 feet deep, and that it was made before the one described by him to the Senate in 1884 was ever thought of! This report, containing this old estimate with this newly-designed jetty, was sent by Col. Mansfield to his superior, General Newton, who, as an advisory member of the Galveston Board, had examined the same estimate in 1880, with the plan of the jetty then contemplated before him, and had referred to its probable cost of $40 per linear foot in that board's report, and whose duty as an advisory engineer of the work necessarily made him familiar with the plans and estimates of it; and he not only indorsed this misleading report by sending it with a concurrent one of his own to the Senate, but positively declared that the 8 000 feet gap or opening between the end of the jetty and the shore, which I had called attention to, did not exist in the plan. It was bad enough to assure Congress that the North Jetty was never located, when his own reports declared that nearly two miles of it had been built before 1880, and that its construction had been recommended by him on a new plan in that year, after that work was destroyed, on the same identical location approved by him in 1874. It was bad enough to deny the fatal defects of the plan in the face of the official proof of their existence, but to lead Congress to believe that $658 525 was the estimate for a jetty from the shore out to 20 feet water, 4¼ miles long, when these officers must have known that it was made for one only about a mile and a half long, will, I think, surpass anything of the kind hitherto recorded in engineering.*

The profession of an engineer is founded upon the grandest system of laws which the human mind is capable of conceiving—a system which is the very embodiment of truth and accuracy, and "which moves to righteousness."

> "Unseen, it helpeth him with faithful hands.
> Unheard, it speaketh stronger than the storm."

These laws not only control the majestic march, and map out the course of every orb in the limitless regions of space, but with equal, and as inexorable precision, align the devious and unseen pathway of the tiniest grain of sand that is swept from the mountain to the sea. No atom is too small to escape their care, and no aggregation of matter is

See Appendix, page 315.

too vast to resist their sway. They are as changless and eternal as their Creator. Applied with judgment and experience, they enable the engineer to control the elements and the great forces of nature for the benefit of his fellow men, now and hereafter; therefore his profession is one of the most elevating and praiseworthy avocations in which the human faculties can be employed. The daily use of laws which never deceive, which weigh with faultless balance each mass and strain and force with which he deals, naturally inculcates a love of truth and fair dealing. The military art justifies and often necessitates deceptive methods, feints, false lights, dummy artillery, and the like; but the civil engineer deals with facts alone, and relies for success upon the ordinances of God himself. Therefore, misrepresentation and deceptive practices are absolutely without resting place or apology in his calling. The moment he descends to use them he puts himself beyond the pale of his profession, and forfeits the sympathy of his associates. He can only use such methods for selfish ends—to hide his errors, or to retain his position at the sacrifice of great public interests. Such an offense carries with it the indubitable evidence that personal interest outweighs, in his opinion, the value of truth, the advancement of science, and the welfare of the public.

18-feet Plans to Produce 25-feet Depth.

Having assured Congress that the width of the jetty channel would be all right, as the North Jetty had never been located; that outlets were never intended to be left in the jetties; that "the project" contemplates raising them above high water; that they only wanted $750 000 to complete the entire thing with the jetty ends in 20 feet of water; that they knew it could be done for that sum, because the South Jetty was built up and completed for its estimate, and that in fact it was really more than the works would cost, and would "leave a margin for dredging to facilitate the scour;" it is not surprising that they should also assert that "the official plan" and estimate contemplated a channel of 25 feet when it never contemplated one of more than 18 feet depth. Accordingly, in his report to the House Committee last January, General Newton says: "The estimate of 1880 * * * contemplated the obtainment of a channel across the outer bar at least *twenty-five feet deep.*" These italics are Gen. Newton's. Col. Mansfield makes the same statement to the Senate Committee March, 1884. This is the first time in the entire ten years after the advisory board sanctioned the Galveston plans, that any of the army engineers officially claimed that they would secure over 18 or 19 feet of water. The following extracts from their official reports, show that the estimates of 1880 *did not* contemplate a depth of over 18 *feet*, and that this claim for 25 feet was simply advanced to defeat my proposition.

On page 1221 of the Report of June 7th, 1880, Gen. Newton declares

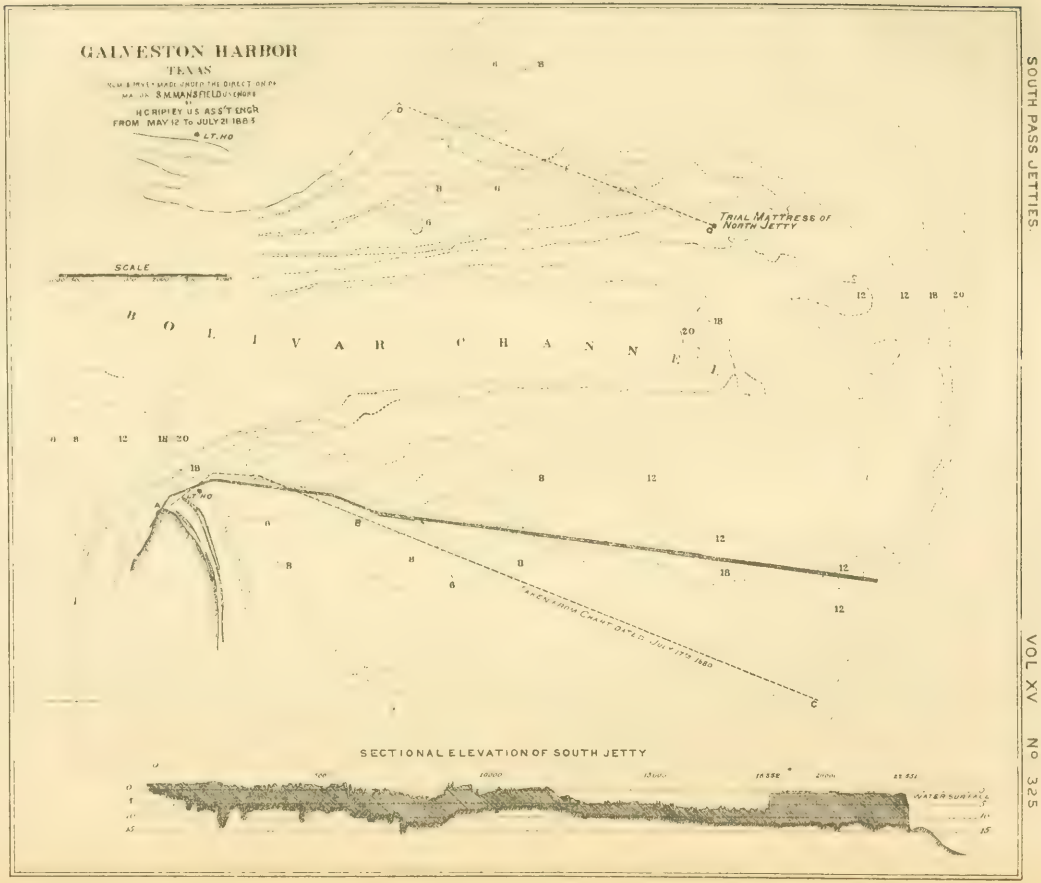


PLATE X
TRANS. AM. SOC. CIV. ENGRS
VOL XV No 325

EADS DISCUSSION ON
SOUTH PASS JETTIES.

"that the project of improving the entrance to Galveston Harbor, as devised by Major Howell, in 1873, had for its object the removal of the inner bar between Fort Point and Pelican Spit, and deepening the outer Gulf Bar *from twelve to eighteen feet, possibly.*" On page 1266 in the second report made (August 9th, 1880), is this statement, signed by Gen. Newton:

"Capt. Howell assumed charge of this harbor in 1869. * * * New surveys were, therefore, instituted by him, and after their completion and a thorough investigation of the subject involved, a project was submitted to the chief of engineers in 1873, for the removal of the inner bar, and deepening the channel over the outer, *possibly to* 18 *feet depth.*"

It will be noted that no change whatever in the location of the jetties was proposed by the advisory board until June 7, 1880, and none since 1880. In the advisory board's report of June, 1880, page 1229, this alteration is thus described: "The South Jetty should incline more to the north, taking the direction indicated by the line *A B*, so as to contract the opening between the two at their outer ends." Accordingly we find, by a chart accompanying Col. Mansfield's Report for 1883, herewith submitted (Plate No. X, page 296), that at a point about 2 800 feet from the Fort Point light-house, the location of the South Jetty as made by Major Howell, and approved by the board of 1874, has been changed more to the north, by which the original width at the outer end of the North Jetty or gabionade, has been reduced to about 10 500 feet, and the parallelism of the two jetties has thus been abolished.

It was not claimed, even in their Report (August 9th, 1880) made three months after this slight alteration was ordered, that the jetties would produce over 18 feet of depth, but, on the contrary, the board, *after this recommendation*, repeat that both piers are "to be extended to such distance as would, by contracting the waterway, deepen the Gulf Bar *to a possible draught of* 18 *feet*" (page 1267). The board, in 1874, and in both of its Reports in 1880, and subsequently, are so guarded in expressing their opinion as to the probable depth that would follow the construction of these embryonic jetties, that it is evident that it thought there was great risk of an important loss of professional reputation in predicting *even* 18 *feet*. This risk was not taken by them until after I proposed to deepen this channel to 30 feet.

The only pretext for this statement is in the report of the civil assistant, Mr. Ripley, to Col. Mansfield, in 1880. But Gen. Newton, with reference to reports of subordinates, says to the River and Harbor Committee, that "these reports have no value, save the authority of their authors." He further says: "They are without any sanction from the proper authority." He now hastily takes advantage of the guess-work opinion of one of the subordinates whom he thus humbles, and boldly declares that the estimates of 1880 contemplated a depth of 25 feet.

We will now turn to page 1213 of the Report of 1880, and see what Mr. Ripley really said about this matter. He says:

"Now, without going into a refined and mathematical discussion, calling for exact data, as to velocity, water slopes and volume of discharge, *which we do not possess*, we will simply point to the facts, that in the present condition, without jetties, there is maintained on the bar, over the area between the two jetties, a very constant depth of eleven feet."

Then, without any "*exact data as to velocity, water slopes and volume of discharge*," Mr. Ripley goes into an arithmetical calculation, and winds up with the statement that "we should look with *confidence* for an extreme depth of not less than 25 feet."

It is not necessary to tell the Members of this Society that Mr. Ripley's conclusions on this subject have no intrinsic value whatever, because of his want of proper data. His superior officers, however, gladly appropriate the report of this humble assistant engineer as though it were their own, and in the next breath Congress is assured that such "reports have no value save the authority of their authors." The following conclusions on this point are inevitable, from a careful study of these reports:

First.—That if the plan and estimate of 1880 were intended to secure 25 feet water, they ought to produce it now without adding 2¼ miles of jetty in 1884 to that plan.

Second.—The fact that the plan of 1880 only contemplated building a jetty one and a half miles long (or 8 090 feet), and stopping it in 12 feet water; while the authors of it present another plan in 1884, involving a jetty four and a quarter miles long, and extending to water 20 feet deep, is proof positive that the authors of the plan of 1880 did not believe, after its defects were exposed in 1884, that it would produce 25 feet depth.

Third.—The fact that Congress was assured in 1884 that the estimate of 1880 was ample to build the North Jetty and secure 25 feet of water, while the plan presented to the Senate and House Committees is a very different one, and far more expensive, is proof positive of the intention to commit Congress, *without its knowledge*, to a much greater outlay than $750 000, and at the same time create the belief in Congress and out of it, by presenting *the old estimate with the new plan*, that my proposal of $7 750 000, for permanent works twice as costly and extensive as those at South Pass, with its guaranty of 30 feet of water, was a huge job; or, as Colonel Mansfield states in his report of March, 1884: "In other words, the Government is to commit itself to the payment of $7 750 000 during the next sixteen years, for about what the Government can secure by continuing the present work during the next two years for $750 000; a work to which it is already committed."

Of course my proposal did not prevent others from underbidding it. With this intention, at the time, some other parties engaged the services of one of the most distinguished and able civil engineers in this Society, O. Chanute, Esq., to visit Galveston and carefully inquire into the probable cost of the necessary works to produce 30 feet of water. Mr. Chanute has told the Society in this discussion, as the result of his visit and

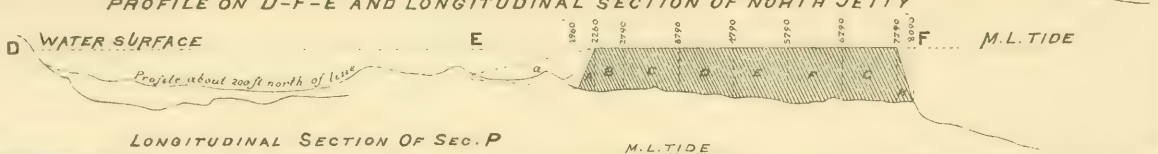
GALVESTON HARBOR IMPROVEMENT

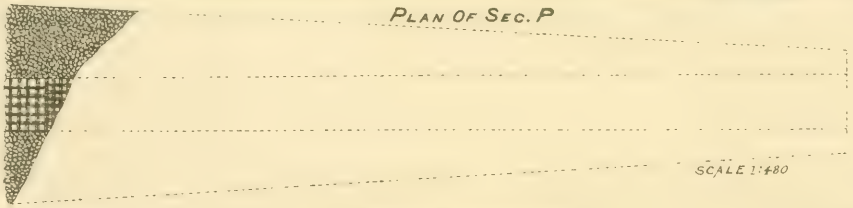
DETAILS OF

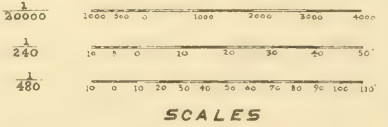
JETTY CONSTRUCTION MADE UNDER THE DIRECTION OF

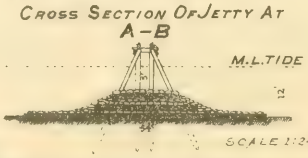
S. M. MANSFIELD, MAJ. OF ENG'RS U.S.A.

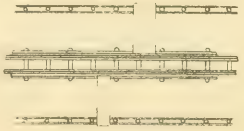
1880











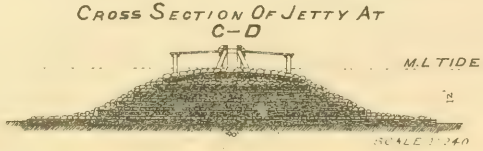


PLATE XI.
TRANS. AM. SOC. C.IV. ENG'RS
VOL XV. No 325

examination, that he found that there was no margin of profit at the price I proposed, and so reported to his employers, and they thereupon abandoned their intention of putting in a counter-proposal.

Alleged Improvement of Channel.

Col. Mansfield says in Col. Merrill's paper, last January:

"Upon the inner bar there is now 26 *feet of water* at low tide where formerly *there was but about nine feet*, and the depth of water over the outer bar has been improved so that navigation has been benefited to the extent of *about two feet*, and this has been accomplished by the Government works, which consist of an *incomplete jetty* running from Fort Point out to the crest of the bar, a distance of 4¼ miles."

In this single sentence I have italicized *four* distinct declarations in direct conflict with official statements made previously by this officer.

The reader is led to believe that 26 feet exists over the inner bar, or shoal between the outer bar and Galveston Harbor. If the outer bar were deepened to 30 feet, 26 feet could not be carried into Galveston Harbor, yet that it can be is clearly the meaning of this statement. That there is 26 feet on the former site of this shoal may be true; but is it frank to make such a statement when no greater depth can be carried into the harbor than existed in 1877, three years before he began his *incomplete jetty?* Then the depth over the obstructing shoal was 21 feet. This fact is proved by the following extract from his Report to the Senate Committee on my proposition, March 18th, 1884:

"The first appropriation for the improvement of Galveston Harbor after adoption of a project, was made in 1874, and work was commenced on the Fort Point gabionade in September of that year, and was continued until June, 1877, having expended for all purposes the sum of $282 000, with the result of effecting an additional permanent increase of 8½ feet in the depth on the inner bar."

This was in 1877, and this brought the depth from 12½ feet to 21 feet. On page 1205 of Col. Mansfield's Report for 1880, he says: "We have now 20 feet of water over the inner bar." This is a foot less than in 1877. His statement leads the reader to believe that before this incomplete jetty was commenced, only 9 feet existed on the inner shoal, while the above quotation from his Report of 1880 proves that he had 11 *feet* more than that at that time. He tells the Senate, March 18th, 1884:

"Work under the present plan was commenced in July, 1880, in the construction of the South Jetty. Upon this work * * * there will have been expended for all purposes the sum of $975 000, including $100 000 donated by the City of Galveston. There resulted from this work, 2½ years from its commencement, an increased depth of 2 feet on the outer bar, which has been maintained up to the present time. There has also been an increased depth on the inner bar due to this work."

If there was an increased depth on the inner bar, there must have been over 20 feet on it. This inner bar had simply shifted its position, and instead of there being an increase of depth on it then, there was 21

inches less than when he began. Plate No. X, accompanying his report of 1883 (and herewith submitted), shows the depth "2½ years after its commencement" to be but 18¼ feet. In this same report he indirectly admits this fact in the following statement: "A shoal in the channel has formed, since the inner bar improvements, inside the bar, with a depth of but 18¼ feet upon it." When the limiting depth into Galveston Harbor, without reference to the outer bar, was but 18¼ feet, how is it "to deal with Congress," to assure it in 1884 that "there has been an increased depth on the inner bar due to this work, which was commenced in 1880," when there was 21 feet on it in 1877, 20 feet in 1880, and only 18¼ feet when he was making this statement?

The part of the South Jetty across the swash channel, where the opening was intended to be left through it, has not been built up high enough to keep the storms from bringing large quantities of sand into the channel, and the result is that neither the location nor the depth of the inner bar are stable. At this time the depth on this shoal is reported to be 21 feet, but as there is but 13 feet on the outer bar, the excess over this 13 feet is of no important benefit. This part of the South Jetty is alluded to in General Newton's Report to the House Committee as something entirely distinct from that jetty, when in fact it is simply part and parcel of the 4¼ miles of that structure. In this Report he says; "The South Jetty, as well *as the works at the inner bar*, have not been of a mere experimental character." He then goes on to repeat the erroneous claim that "those (the works) at the inner bar "have largely increased the depth there."

The official reports prove most conclusively that since this incomplete jetty was commenced, the improvement on the *outer bar* is only 3 inches, or just one-eighth as much as Col. Mansfield reported it to be to the Senate in March, 1884. On page 147, Report of 1880, the Chief of Engineers states that the depth over the outer bar then was twelve and three-quarter feet. On page 1298, Report of 1884, Col. Mansfield reports the depth on it to be but thirteen feet.

Permanence of the South Jetty.

The gabionade system having proved an utter failure in 1879, Col. Mansfield assumed charge in 1880, and he tells us (page 1205 of Chief of Engineers' Report, 1880):

"It is now intended to build the jetties of brush and stone, on a sys-
"tem that undoubtedly will succeed, for it has been applied to open sea
"exposure at the mouth of the Maas, where it has realized all anticipa-
"tions, and established a certain and economical way of constructing
"these sea works on sand coasts."

As the Maas jetties "have not realized all anticipations," and as they are not "an economical way of constructing these sea works," intelligent engineers were surprised, at the time, that unsuccessful works in Holland should be chosen as the pattern for a second experiment at

Galveston, while successful works only a few hundred miles distant seemed to be wholly overlooked. The method of willow construction devised by Mr. Calland was exceedingly ingenious, and as the Maas is a sedimentary river, and, as those works were built up above high tide throughout their entire length, the river deposit soon filled them, and they became, like the works at South Pass, solid and substantial. As the first sea works of the kind known in history, they constitute a splendid testimonial of the ingenuity and ability of Mr. Calland. Unfortunately he made the same error that was committed at Galveston in locating them. They are about 1 000 meters apart, and the frictional resistance of the channel bed is too great for the volume of discharge through them. Instead of producing 26 feet of water, only about 16 feet is maintained by them. They greatly exceeded their original estimate, and a Government commission which examined them about eight years ago, reported that to make them produce the required depth, would more than double their cost at that time. From this it will be seen that an error in locating such works cannot be remedied as General Newton proposes at Galveston, "by further extension into deep water" and "by auxiliary works, if needed," except at immense cost. I am credibly informed that the estimate for *correcting* the mistake at the Maas is over fourteen million guilders. Nearly what I asked for jetties at Galveston, *including* 30 *feet of water*.

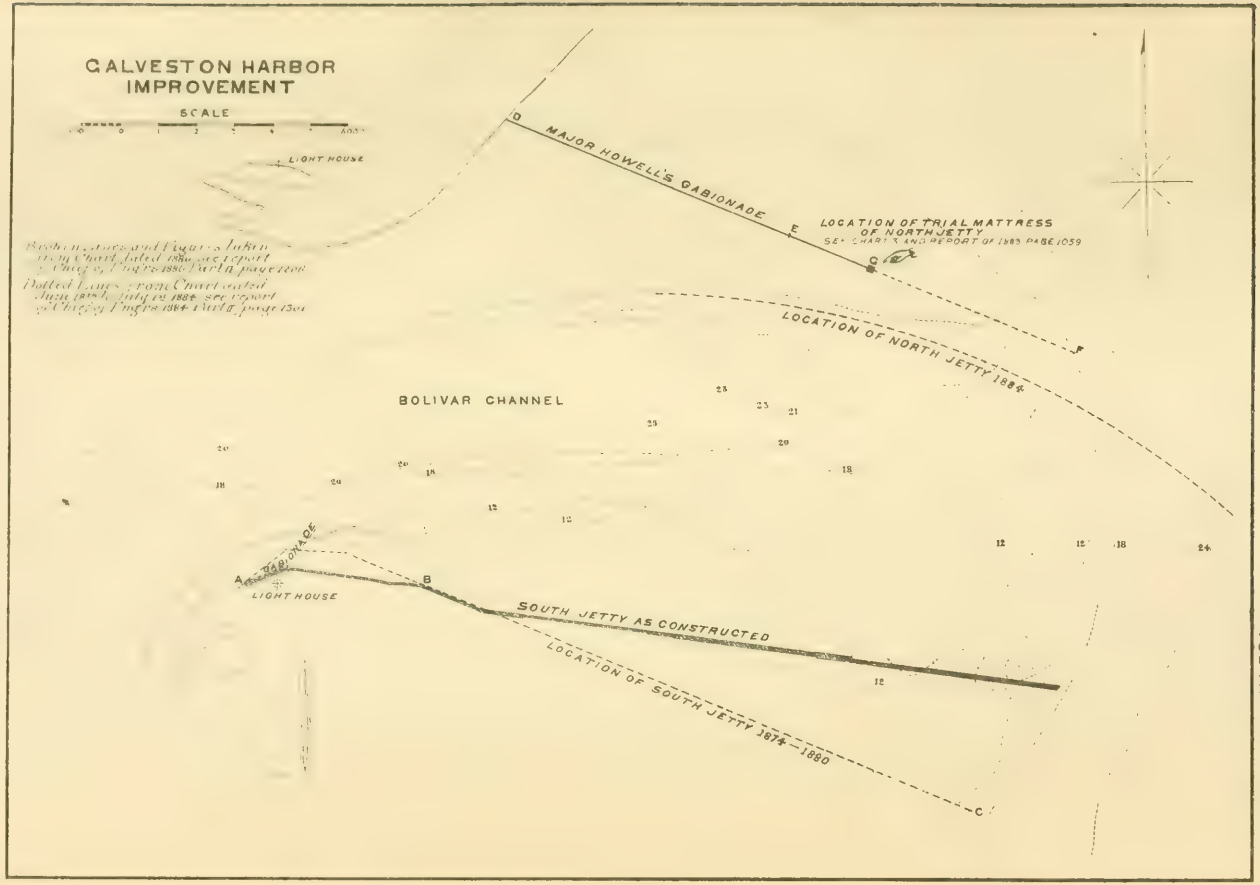
The method of construction devised by Col. James Andrews, Assoc. M. Inst. C. E. (the builder of the South Pass Jetties) and myself, is very much cheaper than that of Mr. Calland. A mattress of a certain size by the Holland system, requiring two days to construct, can by this method be made in less than three hours, with an equal number of men. This method was substantially adopted at Galveston, the Maas system being too expensive.

This second experiment at Galveston has clearly demonstrated that neither of these systems of construction will succeed there, owing to the total absence of river sediment. This is needed to fill up the interstices existing in the brush and stone, to protect the brush from the sea-worm and insure the consolidation of the jetties. The argillaceous or clayey matters in river deposits form a bond which serves to protect the sand particles from being so readily moved through these interstices by the currents induced by the differences of level of the waves and tides on opposite sides of the same jetty. No better evidence of the impolicy of using willows or brush in the Galveston Jetties can be wanted than the fact that the South Jetty, built up throughout to low water in 1883, was found, a year afterwards, to be eight or ten feet lower. The young *teredo* enters the wood through a point scarcely discernible, and grows on what it consumes. Its body frequently attains the length of seven inches, and a diameter of fully half an inch. Its head is provided with a marvellously ingenious apparatus for boring the hardest timber,

and its body is terminated with two fins for propulsion through the water. In warm seas they grow to a size even greater than I have stated.

A full understanding of the real merits of this experiment is not complete without some remarks upon the character of this South Jetty and the no less astonishingly versatile statements made by Col. Mansfield about it. On page 19 I have quoted from his Report of March, 1884, to the Senate Committee, where he says "the South Jetty has *been completed* for the sum already appropriated." In January the declaration of this same officer is read to the House of Representatives (in Col. Merrill's paper) that this jetty is *incomplete*, but has still produced the marvelous effects elsewhere stated. On page 1063 of his Report of 1883, we find a letter from Col. Mansfield to the Mayor of Galveston in regard to what it would take to complete this jetty. Congress had failed to pass a River and Harbor Bill, and Galveston was willing to borrow money to complete the work. Col. Mansfield said then: "$100,000 will keep the work on through the summer, *and will effect the entire completion of the South Jetty*, * * * built up throughout *to the level of mean low water*." He got the money, and, in Col. Merrill's paper, he says: "This structure is a most substantial one, and is not likely to deteriorate much." Two months *before* this statement, Hon. C. R. Breckinridge visited Galveston Harbor and asked Col. Mansfield to go with him in a small boat to sound the depth of water over this "substantial" work. Mr. Breckinridge, while sounding *with Col. Mansfield*, throughout its entire length, found that there was an average depth of between 6 and 7 feet over it at low tide. Eight months before this he had reported it as built up to mean low tide, *and completed*. Only two months after finding it 6 or 7 feet *below* the height he had built it up to, his statement is read to Congress that "this structure is a most substantial one, and is not likely to deteriorate much." Mr. Breckinridge naturally exclaimed: "How is this to deal with Congress, to send report after report, and leave Congress in the belief that this jetty is really substantial, and not likely to deteriorate much? Surely it is not frank and manly in an engineer to hide these facts from Congress and the public." This and other misstatements were exposed in the clearest possible manner by Mr. Breckinridge on the floor of the House, on the third day of February, by quotations from the reports of these engineers; and in reference to them, he says:

"And yet these gentlemen, who are our presumed hope, guides and support—the ones that suffering commerce and a languishing State all look to—they come and certify such things to us as if we would never look up their past reports, but would go on, suffering forever, and forever sustaining them with the people's millions in wretched experiments that the great marine engineers of the world tell us are but child's play. It is in vain for these gentlemen to talk about this as a mere personal question, when the commerce of the country is suffering, * * *


GALVESTON HARBOR
IMPROVEMENT
SCALE
LIGHT HOUSE
Broken Lines and Figures taken from Chart dated 1880 see report of Chief of Engrs 1880 Part II page 1666
Dotted Lines from Chart dated June 1884 July in 1884 see report of Chief of Engrs 1884 Part II page 1360
D
MAJOR HOWELL'S GABIONADE
E
LOCATION OF TRIAL MATTRESS OF NORTH JETTY
SEE CHART AND REPORT OF 1883 PAGE 1059
C
F
LOCATION OF NORTH JETTY 1884
BOLIVAR CHANNEL
GABIONADE
A
B
LIGHT HOUSE
SOUTH JETTY AS CONSTRUCTED
LOCATION OF SOUTH JETTY 1874—1880
C

when they are failing in argument, failing in works, and even failing in statements of fact. I have been astounded at these proceedings." (See *Cong. Record*, Feb., 1885.)

Submerged Jetties.

I pointed out the fact that the waves of the sea would transport the sand from the shoals outside of the jetties over into the jetty channel, and therefore that any improvement of depth which may occur between submerged jetties will be unreliable; and that for the same reason the cross currents, induced by winds or other causes, will carry sand over the submerged jetties and tend to fill up the channel.

Colonel Mansfield says, in his report to the Senate Committee:

"The drifting sand is banked against the north side of the South Jetty, and thus tends continually *to obliterate the jetty channel*. The overflowing water, on the other hand, has excavated a deep trench along the south face of the jetty. The amount of deepening from this overflow varies from 4 to 8 feet, and remains permanent."

According to the reports, an increased depth of only 3 inches on the bar has been secured, while a channel from 4 to 8 feet deep has been created *on the wrong side* of the South Jetty by currents flowing over it.

My argument before the committee conclusively proved the necessity of building these jetties up above high-water mark. Accordingly, we find the following, by General Newton, in his report to the River and Harbor Committee, January 7th, 1885: "As to the height which these jetties should be built, that was regarded by the board as tentative, and it was understood that the height should be finally regulated to meet the requirements of the case." This does not agree with what he said in 1880 (see page 1229, Report of Advisory Commission):

"It has always been assumed that the Galveston Jetties *are to be submerged*, but to what extent can only be determined by results produced from the progress of their construction. *Should an average height of* 5 *feet above the sand bottom prove sufficient* to subserve the purpose for which they were designed, it seems probable that they can be constructed, by either of the methods proposed, for $40 per linear foot."

From this it is evident that the estimate of 1880 contemplated only an average height of 5 feet above the sand bottom for these jetties. The average depth of water which jetties to maintain 30 feet must be built in at Galveston is 14 feet, and they should be 5 feet, at least, above the water. Congress was therefore not only led to believe by these officers that $750 000 would cover the cost of a jetty 2¾ miles longer than the one contemplated by their estimate, but that it would build it 19 feet higher, when the estimate only contemplated its height at 5 feet above the sand bottom.*

Justice to Major Howell's Memory.

General Newton says, in his Report to the River and Harbor Commit-

*A comparative chart (Plate No. XIII) illustrating the progress of the work, and the inconstant features of "the official plan," carefully prepared from official data, is herewith submitted.

tee, January 17th, 1885: "Before leaving this part of the subject, I will state that the gabionade system of Major Howell was never indorsed by the board; but an experiment on a limited scale, by placing a short line of gabions near the bar, and another short line near the shore, was recommended, to test its efficiency." This effort to shift the responsibility of this costly illustration of tentative engineering wholly upon the shoulders of this lamented and ingenious, though singularly unfortunate young officer, whose Fort St. Philip Canal and Galveston plans absorbed so much of the time and study of his illustrious seniors, should not be permitted to pass into history unqualified. It remains, in the general silence which has followed this statement, for one whom he loved but little while living, to shield his memory from this post-mortem reproach. I therefore quote again from official documents signed by General Newton as recently as 1880:

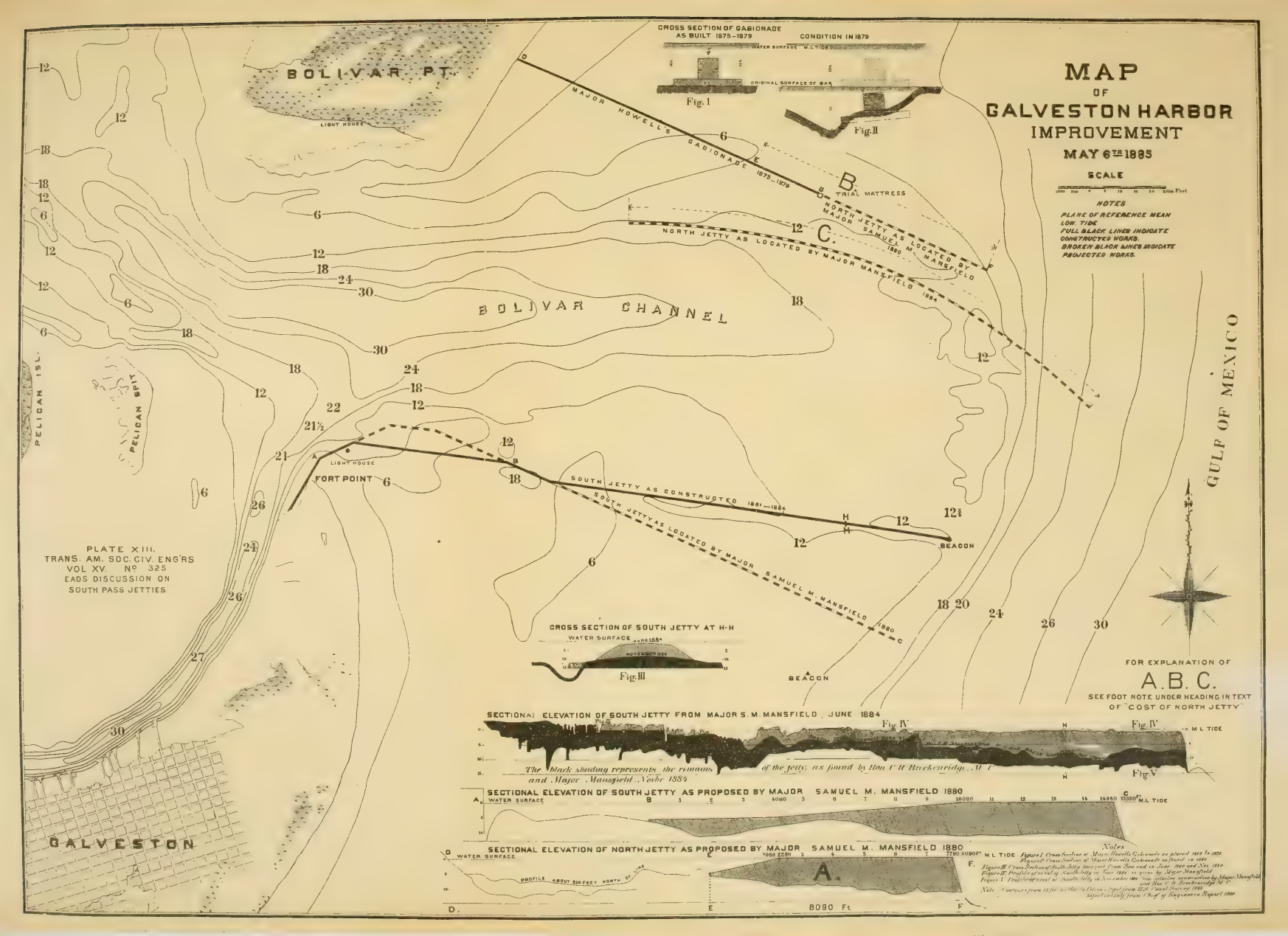
"It was the expectation of Captain Howell, the originator of this method of construction, *as well as of the board that recommended its trial*, that the drift sand of the coast would cover the gabionade. * * * It would be very unjust to the originator of this system of construction to condemn it on imperfect evidence without personal examination. * * * It seems quite probable that the gabions have not been made as strong as they should be; that they are not sufficiently braced inside. * * * Until the above methods have been tried with gabions made stronger * * * the board cannot regard the question of their success or failure as fully decided. It may yet occur that the gabionade will collect drift sand. * * * The time that has elapsed since the Bolivar Pier was commenced" (*the pier that was never located*) "is not sufficient to determine that question."

(See pages 1269 and 1270 Advisory Board's Report of 1880.) Additional official evidence of the approval and indorsement of Major Howell's gabion experiments will be found on page 295, where I have corrected other statements made to the discredit of this zealous young officer.

Tentative Engineering.

General Newton, in his Report to the River and Harbor Committee, strives to defend this policy, and says the jetties of the Mississippi River were placed, originally, 1 000 feet apart, and that I have had this width since diminished. He says: "It is too late, after such an experience, to pretend to regulate beforehand the exact length, height or distance apart of jetties, and thus wholly reject the tentative process."

The Jetties at South Pass were located under a law which, had it not been modified, would have compelled an enormous increase of the flow through the Pass, and for such a channel the jetties would not have been too wide apart. While I had reason to believe that such a channel would not be insisted upon, it was unsafe, when locating the jetties, to assume that the law would be modified. Besides, the South Pass Jetties were built on the most recent deposits of the Mississippi, where it was thought by many engineers that it would be impossible for them


MAP
OF
GALVESTON HARBOR
IMPROVEMENT
MAY 6TH 1885
SCALE
NOTES
PLANE OF REFERENCE MEAN LOW TIDE.
FULL BLACK LINES INDICATE CONSTRUCTED WORKS.
BROKEN BLACK LINES INDICATE PROJECTED WORKS.
CROSS SECTION OF GABIONADE AS BUILT 1875-1879
CONDITION IN 1879
Fig. I
Fig. II
BOLIVAR PT.
LIGHT HOUSE
MAJOR HOWELL'S GABIONADE 1875-1879
TRIAL MATTRESS
NORTH JETTY AS LOCATED BY MAJOR SAMUEL M. MANSFIELD 1880
NORTH JETTY AS LOCATED BY MAJOR MANSFIELD 1884
BOLIVAR CHANNEL
PELICAN ISL.
PELICAN SPIT
FORT POINT
LIGHT HOUSE
SOUTH JETTY AS CONSTRUCTED 1881—1884
SOUTH JETTY AS LOCATED BY MAJOR SAMUEL M. MANSFIELD 1880
BEACON
GULF OF MEXICO
PLATE XIII.
TRANS. AM. SOC. CIV. ENG'RS
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EADS DISCUSSION ON
SOUTH PASS JETTIES
CROSS SECTION OF SOUTH JETTY AT H-H
WATER SURFACE
JUNE 1884
NOVEMBER 1884
Fig. III
FOR EXPLANATION OF
A.B.C.
SEE FOOT NOTE UNDER HEADING IN TEXT
OF "COST OF NORTH JETTY"
SECTIONAL ELEVATION OF SOUTH JETTY FROM MAJOR S. M. MANSFIELD, JUNE 1884
Fig. IV
Fig. IV
M L TIDE
The black shading represents the remains of the jetty as found by Hon C R Breckenridge M C and Major Mansfield Nov'br 1884
Fig. V
SECTIONAL ELEVATION OF SOUTH JETTY AS PROPOSED BY MAJOR SAMUEL M. MANSFIELD 1880
WATER SURFACE
M L TIDE
SECTIONAL ELEVATION OF NORTH JETTY AS PROPOSED BY MAJOR SAMUEL M. MANSFIELD 1880
WATER SURFACE
M L TIDE
PROFILE ABOUT 500 FEET NORTH OF LINE
8090 Ft
GALVESTON

to be sustained. In many places under them we could force a pole down six or eight feet by hand into the soft deposit, and, as a matter of safety from settlement, it was essential to have them as wide apart as was compatible with the object in view.

Col. Merrill and Gen. Newton would lead one to suppose that the price proposed by me for the Galveston Jetties was to be handed over in a lump at once, and the Colonel indignantly asks, how I "or any one else could have completed an $8 000 000 job for $1 000 000"—which was all that Col. Mansfield had. Gen. Newton writes in the same strain about the insignificant amounts voted for Galveston Harbor. This is the flimsiest kind of sophistry. Instead of getting $1 500 000 from the Government, and having only a doubtful 3 inches of improvement to navigation to show for it, we deepened the bar 12 feet at the South Pass before we got a dollar from the Government, and we executed 80 per cent. of the entire work before it had paid us $1 000 000. At Galveston a quarter of a million more than the entire estimate for the works contemplated by the original plan has already been expended, and all that remains now "to point a moral or adorn a tale" of that stupendous folly, are the worm-eaten ruins of a misplaced jetty. For the public treasure we expended at South Pass, the country has had a channel for the last six years through which the largest ships in the world can pass night or day without let or hindrance, and which is maintained by works that will endure longer than the pyramids. The tentative engineering of our adversaries has meanwhile held the commerce of an immensely productive region, far larger than the combined areas of France, Spain, Portugal and Italy, virtually by the throat, helpless, throttled and languishing. Their North Jetty was swallowed up in the sands so long before it was completed, that they actually forgot that it was ever located; and in eight months after its builders had pronounced the other one finished, a ten-foot measure was required to discover the little of it which the worms and waves had left. With a million and a half expended by them, even the location of their works cannot be seen except upon official charts, which they are heartily ashamed of and which they would gladly repudiate.

A distinguished member of the River and Harbor Committee, only eight months after one of these officers had solemnly assured the United States Senate that the South Jetty had been completed up to the level of low tide for its estimate, determined to personally examine this completed work. Guided by this same officer, he actually required a ten-foot pole to tell the depths of water over the shrunken remnant of this four and a quarter miles of completed jetty, for in some places it was then at low tide ten feet to the crest of it. With blistered hands and a sad heart he finished his task, and was rowed back with his official Mentor to the city. The reader can imagine with what astonishment this gentleman heard only two months after this, a declaration

made by this same officer, read to the U. S. House of Representatives, that "this structure is a most substantial one, and not likely to deteriorate much." (See speech of Hon. C. R. Breckinridge, February 3d, 1885, *Cong. Record.*)

It can hardly be expected that I should notice the adverse criticisms which such hydraulic engineers bestow upon the works at South Pass. Those who opposed their construction most bitterly will, of course, be the last to concede the great success which the world accords them. But the public may be deceived by the plausibility of their excuse for not applying the same principles at Galveston which were so successful at the Mississippi. The bar at the latter they say is a *delta bar* formed by a river, whilst that at Galveston is a *drift bar* and created by the sea, hence the same principles are not applicable to each. This is a grave error. Whenever it can be shown that they are not both controlled by the same laws, there may then be a reason for not applying the same principles to each, but not until then. But because the water at one is highly charged with river mud, which is wholly absent at the other, there is a good reason why the same method of constructing the works at one will not succeed at the other. If the reader will examine the official reports of 1873–75 relating to the Mississippi Jetties, he will see that similar arguments were advanced then by Gen. Humphreys and Gen. Newton to assure Congress of the certainty of their failure, namely: because the Mississippi was totally unlike the Danube, and other rivers where jetties had been successful. The simple answer to all this then, was that the same laws control the flow of water everywhere. This answer is equally good in the present case.

The Maas Jetties and other failures are cited to prove that able engineers do not always make successes.

In engineering and other professions based upon exact science, there should be no failures. It has been an axiom with me for more than 30 years that "there are no failures in engineering but what result from bad engineering." As well might we point to some blunder made by an astronomer to prove that he was an *able* one. When an engineer fails to observe any one of the laws that affect the work he has in hand, he will be very apt to be reminded by Nature of such disregard before he gets through with his task. At the Maas Jetties the same mistake was made as at Galveston. They were placed at an enormous distance from each other, the element of friction being wholly overlooked. The gravity of the water and the friction of the bed are the two great forces in all such problems. It is just as unwise to neglect the due consideration of one as of the other. They are like opposite weights in the scales of a balance, and the transporting power of the water is the third great factor, which, by deepening here and depositing there, is constantly striving to maintain a condition of stable equilibrium between them.

Engineers are justified sometimes, by important saving in cost, in

taking certain risks, as, for instance, in not making preparation, when building a bridge, for the recurrence of a flood of extraordinary height, but which is not likely to occur more than once in a score or two of years. If it comes, and disaster follows from its excessive height, the engineer is excusable, especially if he had advised his employers of the facts and acted with their sanction.

But to return to *The Tentative System of Engineering.* Nothing can be more convincing to any man of common sense than that this system should be rigidly enforced by Congress, with limited appropriations so long as the great civil engineering works of the country are planned and executed by military officers. Let us take these Jetty experiments as an illustration. About eleven years ago the plan I proposed for the Southwest Pass was emphatically condemned by Gen. Humphreys, Gen. Newton and others responsible for the Galveston failure. They rejected the theories upon which I based my hopes of success at the South Pass, and very naturally discarded the essential features of the Mississippi Jetties from their Galveston plans. To reduce to a minimum the enormous bar advance which they predicted, they placed their jetties over 12 000 feet apart, overlooking the fact that the friction of this wide expanse of channel bed would neutralize the force of the water discharged from the bay, with which the channel must be excavated and maintained. To facilitate filling the bay, they left enormous openings through the jetties, oblivious of the fact that the slope of surface must *at first* be increased before any channel deepening can take place, and that the flow into and from the bay must be obstructed—not facilitated—to cause this increase of slope. They also overlooked the loss of excavating force in the channel by the escape of the ebb tide through these openings, as well as the inevitable transportation of immense quantities of sand into the channel through them. Then they decided that it was not necessary to extend their jetties out to deep water, forgetting that the waves of the Gulf are continually heaping up the sands on to the shore in lesser depths than 30 feet, and will thus make any channel through the shoals beyond the jetties unstable. Then they determined to keep the jetties below the surface of the water, ignoring a fact which is illustrated on every sand shoal in the world on which an obstruction of any kind is to be found, over which a current of water flows; namely, that on the lee side of such obstruction there is found a depression in the sand caused by what engineers term the overfall of the current, which makes the undermining of the jetties simply a question of time unless built with enormous width of base.

Ships wrecked on a sandy beach disappear under the sands from this cause. Forced broadside on shore by the storm, the overfall of the storm waves which pass over it excavates a deep trench on the shore side of the vessel, while that part of the waves which does not pass over is deflected as "racers," and rushes along the sea side towards the stem

or stern, excavating a deep trench on that side also. In this way the wreck is undermined on both sides until it sinks below the general level of the sand, where none but the most violent storms can produce any further deepening. Then, after the subsidence of such a storm, the ordinary wave action which is always transporting sand up on to the sea beaches, fills these trenches, levels off the bottom, and all signs of the ship disappear forever. As the overfall changes with the wind during the severe storms to which jetty works are exposed, these trenches are formed on each side of them and they do not require to be very deep to undermine the jetty.

As the action of these secondary currents and overfalls on sandy bottoms are matters of such every-day observation to those who are familiar with water courses, sea beaches, and harbors, and especially to hydraulic engineers, the astonishment with which these officers learned the fact that "contrary to expectation," their long line of submerged gabions was actually standing in a very dilapidated condition in a deep trench, is one of the most amusing incidents connected with the lavish expenditures which Congress has been making for so many years to enable these officers to acquire some practical knowledge of hydraulic engineering. A few more extracts from these official reports will serve to show a little of the really funny side of this particular investment of a half million dollars in their supplemental education We will first describe very briefly this North Jetty which the sea so soon buried beneath its sands.

This gabionade, which extended nearly two miles out to sea, may be compared to a row of huge bricks laid flatwise, with their ends against each other. The gabions were made somewhat in the form of a box, twelve feet long, six feet deep, and six feet in width. The frame-work was willow, and this was plastered over with hydraulic mortar to the thickness of about three inches. They were sunk on the sand, and afterwards sand was pumped into them, and covers placed on them to keep it from washing out. At one time the wind drove the sea from the north over this row of bricks, and, as a matter of course, the overfall excavated a trench along the south side of it. When the wind came from the south, the current was driven over it in the opposite direction, and excavated a trench on the other side of it. Any man not an engineer, but familiar with the action of currents on sand bars flowing over obstructions, would have known that this would occur, and that four of the highest officers of the Engineer Corps of the Army should approve of *even experimenting* with such a structure is one of the most amazing things in the history of engineering.

On page 603, Chief of Engineers' Report, Part 1st, for 1879, will be found the following:

"They have all settled quite uniformly. This settlement during the September storm amounted to about four feet, leaving the top of the

gabionade but little above the surface bottom; therefore, the gabions of this line will hereafter be placed upon mats. Work was, therefore, confined to the extension of the Bolivar pier gabionade over the outer bar (North Jetty); 354 gabions were sunk during the year, leaving at its close a gabionade over the outer bar 7 332 feet in length * * * Work was suspended December 1, 1877, and not resumed until June, 1878. * * * There were a large number of gabions left on hand which were used in construction of the gabionade when work was resumed. Of these, 121 were washed away during the past year."

Major Howell, in the same volume, page 911, says, "at the close of the year it was ascertained that *the gabions with mats had sunk* as much as those without, but not so rapidly."

The Chief of Engineers, General Humphreys, October, 1879, informs Congress that the surveys of the officer in charge, Major Howell, "do not show any change on the outer bar that can be attributed to the 7 332 feet of gabionade then built." As the first 4 000 feet had already sunk their full depth into the sand, and the gabions with mats under them had sunk as much as those without, this statement of the Chief of Engineers is remarkably "child-like and bland."

Major Howell says on page 911 of the same volume: "The amount of sinking ranged from three to seven feet, and, contrary to expectation the gabions did not become covered with sand, but remained standing in a trench and entirely uncovered." That is the entire 7 332 feet of the North Jetty which Col. Mansfield, Col. Merrill and General Newton assured Congress had never been located, had by this overfall action of the current been lowered down level with the original sand bar on which it was built, into a deep trench. On page 109 of this volume, the Chief of the Corps informs Congress that "surveys of the outer bar do not show any change that can be attributed to the influence of the gabionade, and that such change cannot be expected *until it is extended further out.*"

This remark of the head of the corps will excite the risibles of every engineer who reads it; nevertheless, *as he recommended it,* the requisite funds were voted by Congress, and the expensive farce was continued until this experimental North Jetty extended out 10 220 feet from the shore (see page 1225, Report of 1880).

Can anything be more suggestive of the value of tentative engineering? Imagine four of the oldest, ablest and most astonished officers of the Corps of U. S. Engineers, sitting in earnest consultation over this incorrigible and moribund North Jetty, that was never located! And two of these were authors, too, of elaborate treatises on the jetty system, for the full text of which see Chief of Engineers' Report, 1874, pages 833 to 887; also Appendix M, page 678, Physics and Hydraulics of Mississippi River, Humphreys and Abbot.

The six army students engaged in the study of tentative engineering at Galveston included the then chief of the corps; the last chief of the

corps; the present chief of the corps; the oldest member of the corps; the original inventor of the gabionade and submerged jetty and outlet system, and the author of a new method of estimating the cost of public works, designed especially for the use of Congress. The Senate Committee, in their report on Bill 1652, declares that this gabionade experiment cost $527 000, and $975 000 more was expended on the second experiment, viz.: in building a brush and stone jetty that shrunk to half its height within a year after its completion. And these failures occurred after having all of the erudition contained in the treatises of Generals Humphreys and Newton on the jetty system to instruct them! Eminently successful jetty works too were within 300 miles of them! With these extraordinary advantages, these tentative students actually failed to learn in ten years that excessive widths between jetties; openings in them to facilitate the tidal flow; submerged jetties; and jetties that stop in shallow water on top of the bar, are each and all a violation of the very essence of the jetty system. The official quotations I have given, show conclusively that these four important facts were totally unknown to them until I pointed them out to the Congressional Committees in 1884. And now, after a patient, confiding and generous Government has given them about a quarter of a million dollars more than their original estimate of 1874 for the entire work (see page 735, Report of 1874), it does seem a little ungrateful on the part of Gen. Newton and these two Colonels to upbraid it, even in respectful language, for not making the appropriations for these absurd experiments more rapidly. The failure of "the official plan" is all owing to dilatory appropriations, and that of the gabionade is put upon poor Major Howell.

"Alas for the rarity of Christian charity under the sun."

General Newton, defending the "tentative process," and as an apology for the slight deepening claimed at Galveston, says in his Report (1884) to the House Committee: "You will not find in the whole world an engineer who could calculate within a foot or two what the effect of the jetties at Galveston would be, while the very attempt to do so would probably prove that the person was incompetent."

Never was a gallant soldier more cleverly "hoist with his own petard," as the reader will see by turning to pages 844–5 of the Report of Chief Engineers, 1874, where, in his elaborate essay before mentioned, intended to defeat my proposal to deepen the mouth of the Mississippi, Gen. Newton not only calculated "*within a foot or two* what the effect of the jetties would be" at each mouth of the river, but carried his numerical studies to such a nice point that he actually foretold it to the *fraction of a foot.* The annual rate of shoaling which would occur in case they were applied to either pass, he found by arithmetic to be 1 foot at Southwest Pass, 2.16 feet at Pass à l'Outre, and 3¾ feet at the South Pass. The annual rate of bar advance at each pass under the

stimulating effect of the jetties was also carefully ciphered out by him "within a foot," and was found to be for each of the three, 1 014 feet, 1 208 feet, and 2 240 feet, respectively. Of the South Pass he said: "Its narrow width between jetties, only 660 feet, would soon be *consumed by the convergence* necessary to keep up the scour." That is, the jetties were to give it the galloping consumption at the rate of 2 240 feet a year. He said also, "the Pass at the present time (11 years ago) is hanging between the conditions of a *live pass* and a *stagnant ditch*, to the last of which results it must arrive if a revolution in the delta does not redeem it. And this most probable fate will be precipitated by applying the jetty system to its mouth."

The jetties were applied to its mouth, and according to Gen. Newton's calculations they would now need to be extended over two and a half miles beyond their present terminals, or else the channel would be twenty-two and a half feet shoaler than it was six years ago when we first secured 30 feet through it. As they have not had to be extended, but are 300 feet shorter than they were originally designed, and as the channel is better and deeper than ever, it nevertheless has not "proved that the person was incompetent" who made these calculations. If he had been a civil engineer it would undoubtedly have proved him very incompetent, and people having their own money to spend and knowing this, would not have been likely to select him to plan their harbor works. But the case is quite different with military engineers whom the Government is anxious to have instructed in these matters, so as to save the country the expense of employing civil engineers except as subordinates to them. Yet this is the only civilized nation in the world that places its hydraulic works under the charge of its military engineers.* Other countries have not yet imitated us in it, as they have doubtless seen that most of the money expended by the United States has been paid for experiments designed to give to the officers in charge "a much needed light" (as Gen. Newton frankly expressed it) in this kind of work. It is probable that the immense sum spent in the futile effort to prevent the caving of the river banks at New Orleans, with exaggerated window screens, made with hand-woven fishing canes, and hung with sand bags, would have paid the salaries of half a dozen competent civil engineers for five years, and have prevented this expensive illustration of experimental or tentative engineering. The cost of the ever-memorable Galveston gabionade would have paid for a bureau of competent civil engineers until the advent of the 20th century. Scores of these expensive and unsuccessful experiments made at the Government expense, can be found by examining the Official Reports of the Chief of Engineers during the last

*Mr. Corthell has in this discussion shown by the letter of James Forrest, Esq., Sec. of the Institution of Civil Engineers in London, that Col. Merrill was mistaken in his statement that military engineers in England are employed on such works.

two decades, but they have not proved any incompetency yet. If these jetty calculations had proved this officer incompetent, a wise government would not have placed him afterwards at the head of his corps, in supreme charge of all the harbor improvements of the United States. He is now chief adviser of the Government on all questions of hydraulic engineering, and the members of the advisory board of Galveston Harbor under him still have, as they have had for many years, more or less direct or supervisory control over all of our seaboard harbor works.

The sequel has proved that the calculations and predictions of these officers respecting the South Pass and its jetties were simply absurd professional mistakes, and that if their advice had been followed, enormous expenditures for an unfinished canal would have been made, while the immense commercial benefits which the jetties have conferred upon the whole country for the past six years would have been unknown. The gravity of these errors has been intensified by the failure at Galveston, where the mistaken theories which led to them have been tenaciously adhered to until this day, notwithstanding the practical demonstration which the South Pass Jetties gave six years ago of their unsoundness. In the whole history of this Government in its dealings with the officers of the army, there is nothing in such prominent contrast as its treatment of the errors of the officers of the Engineer Corps and those of the other corps of the army. Mistakes or lack of judgment on the field in the heat of the contest, when time for investigation and deliberation could not be had, have been rewarded with courts martial, loss of pay, of rank, or disgrace; whilst notwithstanding absolutely inexcusable and frequent blunders made by several of the officers of the Engineer Corps after abundant time for investigation, and involving immense loss of public money and vexatious delays to great and much needed improvements, they have been rewarded with promotions, a continuance of official confidence, and the disbursement of still more lavish appropriations.

Colonel Merrill says: "It seems to me that a wise and prudent engineer, dealing with such a difficult problem as the improvement of a harbor on a sandy shore, should proceed cautiously, feeling his way by degrees, prepared to change his plans whenever their effect seemed unfavorable." This course would certainly be the way a wise and prudent engineer should proceed if he were not familiar with the laws involved in the problem. Here we are mutually agreed. He continues: "I am disposed to believe that, as a rule, jetties should be above the level of the sea; * * * but in spite of this belief I should certainly try first to build two submerged jetties, and see how they worked, because it is quite possible that one or "the other might require removal." This is quite possible. At all events, one who knows nothing about such problems "should proceed cautiously." If

the Government would utilize its army surgeons, and put them in charge of our harbor works, the wise and prudent ones would proceed with like caution, and no doubt a success would attend them quite equal to that which has resulted from the ten years of labor and the million and a half expended at Galveston. I do not believe, however, that the surgeon would, when called on to amputate a man's leg, adopt the tentative system; and, "feeling his way by degrees," cut off a little of it to-day and a little more to-morrow; "prepared to change his plans" if he found he had got hold of the wrong leg, and feared that "the other might require removal." Nor do I think that the Chief of Engineers, or Colonel Merrill, would proceed on the tentative system if required to construct such fortifications as would check or turn the course of a victorious and advancing foe. No one in the land would feel more certain than I should, that either one of these two accomplished military engineers would make a perfect success of such work, and that neither would hesitate for a moment, after a proper study of the location, as to the construction and position of every detail of it from moat to citadel. But it is little short of cruelty on the part of the Government, to impose upon its gallant and worthy military officers such problems as are presented in the improvement of our rivers and harbors, and which require long and practical familiarity with the forces and phenomena developed by waves, tides and currents, to insure their economic and successful solution. Our naval officers could be educated to master such problems at much less expense, and, I believe, far more successfully. An amphibious idiosyncrasy is rarely met with in the army. Without it the army engineer will be a failure as an hydraulic engineer, and when put at it by an injudicious government, will run great risk of having his laurels transformed into sea-weeds. If Todleben himself had been put in charge of Galveston Harbor ten years ago, instead of the defenses of the Crimea or Balkans, he would probably have made as miserable a failure of it, and as costly an experiment to the Government, as the four highest officers of the Engineer Corps, and their two subordinates, have made at Galveston.

> "One science only, will one genius fit,
> So vast is art, so narrow human wit."

Appendix.

Reports of General Newton and Colonel Mansfield, submitted to the Senate Committee on Commerce in March, 1884, on Senate Bill 1652.

Note.—The following reports, to which reference is made under the heading of "Cost of the North Jetty," in the text, are added as an *Appendix*, because they do not appear in the Annual Report of the Chief of Engineers. The portion of these reports sustaining the asser-

tions made by me, under the above heading in the preceding review, have been italicised by me. JAS. B. EADS.

WAR DEPARTMENT,
WASHINGTON CITY, March 29th, 1884.

SIR,—In returning herewith Senate Bill No. 1652, a "Bill to Provide for the Improvement of the Channel between Galveston Harbor and the Gulf of Mexico," which, with others, was referred from the Committee on Commerce for information, and for the views of this department thereon, I have the honor to state that the subject was referred to Major S. M. Mansfield, Corps of Engineers, in charge of the improvement of Galveston Harbor, and to inclose herewith a copy of his report dated the 19th instant, which contains much information upon the subject in question.

I inclose also the report of the Chief Engineer submitting Major Mansfield's report to this department, and beg to invite especial attention to his views as therein presented.

Very respectfully, your obedient servant,

ROB'T T. LINCOLN,
Secretary of War.

Hon. S. J. R. MCMILLAN,
Chairman Committee on Commerce,
United States Senate.

OFFICE OF
CHIEF OF ENGINEERS UNITED STATES ARMY,
WASHINGTON, D. C., March 27th, 1884.

Hon. ROBERT T. LINCOLN,
Secretary of War.

SIR,—Senate Bill 1652, Forty-eighth Congress, First Session, "To Provide for the Improvement of the Channel between Galveston Harbor and the Gulf of Mexico," referred to this office on the 6th instant, is herewith respectfully returned. It was referred from this office to Major S. M. Mansfield, Corps of Engineers, in charge of the improvement of Galveston Harbor, who has made a report thereon, copy submitted, to which attention is invited.

The bill in question is drawn in favor of James B. Eads and associates, as the proposed constructors of works for the purpose of obtaining an increased depth in the channel between the Gulf and the Harbor of Galveston.

This bill provides for the payment of $7 750 000 to the said Eads and associates for the construction of said works, and for obtaining a navigable channel at least 30 feet deep, when measured at mean high tide. Section 5 also provides for the payment of $100 000 annually for a period of ten years for the maintenance of the channel.

For the first 2 additional feet of depth obtained in the channel, Mr. Eads and associates are to receive $2 000 000—that is, $1 000 000 for

each additional foot, and for each additional foot until the depth of 30 feet be obtained, a smaller and varying rate of payment, and when said depth is obtained such sum of money shall be paid as shall, when added to the amounts before paid, make up the sum of $7 250 000. The remaining $500 000 of the total compensation for obtaining the full depth of 30 feet in the channel, shall be deemed to have been gained by said Eads and his associates, but the same shall remain in the possession of the Government for a period of ten years, and during this period they shall be entitled to interest at the rate of 4 per centum; and if they shall have maintained that depth for the full period of ten years, they shall receive the retained sum of $500 000, making a total (exclusive of interest and of $1 000 000 for maintaining the depth of 30 feet for the ten years), of $7 750 000. This much for the proposed relation of Mr. Eads and his associates to this improvement.

It is supposed that the main reliance of the parties seeking this contract will be upon the effect of jetties in obtaining improved depths at the entrance of rivers and other bodies of water emptying into the ocean. The effects of these constructions under the varying circumstances in which they may be applied, are well known to those engineers who have been engaged in the improvement of bars. Congress at an early date in 1825 commenced the grant of appropriations for constructing jetties on the lake coast, which were expended under the direction of the United States engineers with great benefit to navigation.

The mode of improvement at Galveston has been reported upon by boards of engineers in 1874–75, 1876–80, and two jetties, a south and north, were recommended as necessary, and this view has been since held without a change by this office.

Up to the present moment but one jetty has been constructed by Colonel Mansfield, viz.: the South Jetty—but a slight improvement of depth is claimed, and the cause becomes manifest from a casual inspection of the chart, which indicates very clearly the necessity for the additional construction of the North Jetty.

Colonel Mansfield states that $975 000 *has been expended upon the construction of the South Jetty, and that the construction of the North Jetty will cost* $700 000. That this construction will give more than an additional 2 feet of depth in the channel connecting the Harbor of Galveston with the Gulf, is entirely certain.

The case can be plainly stated as follows: Colonel Mansfield, with the expenditure of $700 000 *and two seasons' work, will obtain an increase of depth exceeding* 2 *feet, and probably reaching* 5 *feet or* 6 *feet*, while Mr. Eads and associates promise, after a period of two years and eight months after the passage of this bill, to gain 2 feet in depth for an expenditure on the part of the Government of $2 000 000.

It is hard to understand how the existence of the South Jetty, but just now constructed by the Government, and which can be made an

important factor in obtaining an early and marked increase in depth, should have been entirely ignored in this bill.

The North and South Jetties placed according to the official plan, can, by further extension into deep water, and by the construction of auxiliary works, if needed, be made to develop all the depth of channel which the nature of the locality will admit.

In conclusion, the present moment appears premature for the passage of the bill in question, and it is thought to be contrary to sound policy to interfere with the progress of the works at Galveston, which are now so far advanced that every additional dollar expended may mark an increased depth in the channel. Should Congress be resolved to make a change in the administration of this work, at least it will be for the interests of the service to await the construction of the North Jetty, and the observation of its effects, which will lead to the possession of facts and data calculated to throw a needed light upon the amount necessary to be expended for obtaining a proper depth on the bar.

Very respectfully, your obedient servant,

JOHN NEWTON,

Chief of Engineers, Brigadier and Brevet Major-General.

UNITED STATES ENGINEERS' OFFICE,
HENDLEY BUILDING, GALVESTON, TEXAS, March 18th, 1884.

To the Chief of Engineers United States Army, Washington, D. C.

GENERAL,—Senate Bill 1652, to provide for the improvement of the channel between Galveston Harbor and the Gulf of Mexico, referred to me to report by first indorsement on wrapper from your office, is hereby returned, and I have the honor to submit the following, beginning with a brief history of the work:

First.—The first appropriation for the improvement of Galveston Harbor was made in 1870. This and the two succeeding appropriations, amounting in all to $76 000, were expended in plant for dredging the inner bar and building and strengthening the pile breakwater before any plan for the improvement of the harbor had been inaugurated. This work effected a deepening of about 3 feet on the inner bar.

Second.—First appropriation for the improvement of the harbor after adoption of a project was made in 1874, and work was commenced on the Fort Point Gabionade in September of that year, and was continued until June, 1877, having expended for all purposes the sum of $282 000, with the result of effecting an additional permanent increase of 8½ feet in the depth on the inner bar.

Third.—Work for the improvement of the outer bar was commenced in April, 1877, by the commencement of the construction of the Bolivar Gabionade. This work was continued until November, 1879, when it was abandoned before reaching the bar and without appreciable results, having expended upon it for all purposes the sum of $245 000.

Fourth.—*Work under the present plan was commenced in July*, 1880, *in the construction of the South Jetty. Upon this work, with the exhaustion of the small amount now remaining, there will have been expended for all purposes the sum of* $975 000, *including* $100 000 *donated by the City of Galveston. This sum accomplished the building of a jetty* 4¼ *miles long, and varying in height from* 2½ *to* 14 *feet.* There resulted from this work, two and one-half years from the date of its commencement, an increased depth of 2 feet on the outer bar, which has been maintained up to the present time. There has also been an increased depth on the inner bar due to this work. On the site of the inner bar there is now a least depth of 25 feet where there was but 11½ feet in 1875, after the great storm of that year. A shoal in the channel has formed since the inner bar improvement, inside the bar, with a depth of but 18¾ feet upon it. But this is due to the decay of the pile breakwater, which, when in repair, prevented the sand from drifting across the point into the channel.

Fifth.—An increase of only two feet on the outer bar at first view seems very little, considering the magnitude of the work accomplished, and to some may raise a doubt as to the ultimate success of the work; but a careful study of the changes continually going on, and the causes which produce them, will explain why there has been no greater increase, and also furnish the proof of the ultimate success of the work. The efficient causes which are producing constant changes upon the bar are :

1. The ebb and flow of the tides.
2. The littoral current.
3. The translatory power of the seas from the Gulf.

The average duration of the ebb current spring tides is about six hours; that of the flood current and slack water about eighteen hours.

The littoral current flows with varying strength in a direction parallel to the shore line either east northeast or west southwest, but its prevailing direction, and the one in which it has the greatest strength, is west southwest.

The facts, then, are as follows : The axis of scour due to the ebb current, unaffected by other forces, would be in line of the axis of Bolivar channel. But the South Jetty, not being of sufficient height to prevent the water from flowing over it, the littoral current deflects the axis of scour either to the north or south, and by an amount proportional to the relative strengths of the two currents. It will thus be seen that the axis of scour is continually shifting, and hence no concentration of energy. During slack water the littoral current flows along the bar approximately at right angles to the axis of scour during the ebb, and drifts the sand into any depressions which may have been produced by the previous ebb. This drifting sand is banked against the north side of the South Jetty, and thus tends continually to obliterate the Jetty channel. The overflowing water on the other hand has excavated a deep trench along the south face of the Jetty. The amount of deepening due

from this overflow varies from four to eight feet, and remains permanent. A similar result followed the construction of the Bolivar Gabionade, the Brazos Ridge and Aransas Jetties. In each case a deep trench was developed along the south side, showing the same cause, producing the same effect in each instance.

The effect of the flood tide upon the bar is probably very small, unaffected by other forces. It flows toward the gorge in lines natural to the crest of the bar, and hence is wanting in concentration, so essential to beneficial results. The effect of the sea is to heap up the sand upon the crest of the bar, and push it in toward the gorge.

Sixth.—A knowledge of the causes and their effects in producing the changes enumerated above, enables us to give to the North Jetty such a form and location as will utilize to the fullest extent the forces essential to the accomplishment of deep water across the bar, in accordance with the plan already adopted.

To this end we would give the Jetty the form of a curve, concave toward the channel, whose radius is 23 000 *feet. The tangent of this curve at its river end should coincide with the twelve-foot contour on the north side of Bolivar channel, and its inner end should be abreast the twenty-seven-foot depth in that channel. The outer end should extend to the twenty-foot contour. This would give it a length of* 14 340 *feet, and, on the basis of the cost of the South Jetty, can be built twelve feet high for the sum of* $700 000. By commencing at the outer end, and constructing that portion upon the shoalest part of the bar first, we would accomplish results which would at once increase the available navigable depth across the bar.

Seventh.—It is plain to see what effect such Jetty would have. The sand, drifted along by the littoral current, would be banked up against the north side, and the overfall, due both to the littoral current and the flood current, would excavate a deep trench along its channel face. The ebb current would be directed into a single channel along the concave face of the Jetty, and would be concentrated in such a manner as to exert its maximum effort upon the immediate crest of the bar, and, in accordance with the universal law that deep water always exists in the concave bends of flowing streams, we should expect as great a depth along the face of this Jetty as in the straight channel abreast its inner end. This form and location would also be effectual in protecting the channel from east and northeast storms, and by extending it gulfward on the curve proposed, together with the extension of the South Jetty, any desired contraction of the outer ends can be obtained.

Under the present system of work, adopted at the close of the year 1880, *the estimate for the entire completion of the two Jetties for confining the outflow of Galveston Bay, looking to obtaining twenty-five feet of water, were: For the South Jetty,* $1 080 448; *for the North Jetty,* $658 525; *for plant,* $40 000, *and contingencies,* $46 840, *making a total of* $1 825 813. *These estimates, as stated, were made upon the supposition that the work would be*

continuous, and \$500 000 *was the sum asked for annually. A little over three years would then have been required for the entire completion of the work. The only criticism, then, to be made upon the work, is that the appropriations have been insufficient in amount to permit its completion within the specified time. The South Jetty has been completed for the sum already appropriated, the plant is now in hand, and we have the experience that will certainly enable us to complete the North Jetty within the estimate, and still leave a margin for dredging to facilitate the scour if it should be found necessary or desirable to hasten it; and not over two seasons will be required for the expenditure of this money, and it is reasonable to expect by that time will be solved the vexed question of deep water on Galveston Bar.*

Now, what is proposed by the bill under consideration? No plan of operations is submitted in it. The Government is asked to furnish Mr. Eads with a survey of the harbor upon which it is supposed plans will be perfected. It is inferred from the mention of jetties, etc., that the same system of contracting the outflow between jetties that is now in progress, is to be followed by Mr. Eads. Eight months are allowed him in which to begin operations, and twenty-four months more are allowed him in which to put two feet more water on the bar, for which the Government is to pay \$1 000 000 per foot, thus providing for an additional two feet of water in two years and eight months time from the passage of the bill. Upon the obtaining of another foot of water the Government is to pay \$500 000, and the next foot \$500 000, and so on during sixteen years, in which Mr. Eads is allowed to put thirty feet of water on the bar at high tide. The only guarantee that the work will be carried out is, that the Government may at any time annul the contract by Special Act of Congress. *In other words, the Government is to commit itself to the payment of* \$7 750 000 *during the next sixteen years, for about what the Government can secure by continuing the present work during the next two years for* \$750 000—a work to which it is already committed. What is to be gained by this procedure I am unable to say. I may be allowed, however, to express my conviction that the passage of this bill is not warranted by the facts, and that in the single respect of deep water it is entirely uncalled for.

Very respectfully, your obedient servant,

S. M. Mansfield,

Major of Engineers, Brevet Lieut.-Col. U. S. A.

Col. William E. Merrill, M. Am. Soc. C. E.—I have requested the privilege of adding a few remarks, because Mr. Eads' paper introduces much new matter, and for the additional reason that he charges me with having made erroneous statements; this is my apology for again troubling the Society on this subject. The publication of my previous paper in the *Congressional Record* was made without my consent, and was as great a surprise to myself as to any one else. In this connection it

may be worth stating that Mr. Eads' paper appeared in the New Orleans *Times-Democrat* of November 30th, 1885. If the accidental premature publication of my remarks is reprehensible, the intentional publication of his reply is more so.

Mr. Eads' original contract with Congress may be tabulated as follows:

Contract dated March 3d, 1875, with James B. Eads, for construction of South Pass Jetties.

Channel to be Obtained.	Cash Payment.	At End of 12 Months.		Annual Payments for 20 Years.	Withheld for 10 Years, U. S. paying Interest.		Withheld for 20 Years, U. S. paying Interest.	
		Principal.	Interest.		Principal.	An. Int.	Principal.	An. Int.
20 × 200	$500 000							
22 × 200	500 000							
24 × 250	500 000	$250 000	$12 500					
26 × 300	500 000	250 000	12 500					
28 × 350	500 000	250 000	15 000					
30 × 350	500 000	500 000	25 000	$100 000	$500 000	$25 000	$500 000	$25 000
Total..	$3 000 000	$1 250 000	$65 000	$2 000 000	$500 000	$250 000	$500 000	$500 000
			Grand Total.....					$8 065 000

I have tabulated the items in the precise order in which they appear in the Act of Congress. Mr. Eads says that I ought to have omitted the $2 000 000 for maintenance. That is a matter of opinion, and I leave the facts to speak for themselves. The item for maintenance is one of the leading features of the contract, and it cannot be omitted in summing up the "total" cost, which is what I gave. As a matter of fact, I really underrated the amount involved, by omitting the large sums to be expended in interest. I have now included these sums, and the table herewith is an exact summary of the Act of Congress.

In 1878, Mr. Eads again went to Congress and procured the passage of an act authorizing a payment in advance of $500 000, that would be due as soon as a channel 24 by 250 had been obtained, and also authorizing a further advance of "such sums, not exceeding in the aggregate the gross sum of $500 000, as he, or they (his assigns), may require in the prosecution of the works authorized by said hereinbefore recited act, to pay for materials furnished, labor done, and expenditures incurred from and after the passage of this act, in the construction of said works." In other words, he succeeded in getting the United States to furnish the money needed to carry on his contract.

In 1879, he again went to Congress, and had the unfinished part of his contract set aside, and new conditions made, as shown in the following table.

Amended Contract of March 3d, 1879, with James B. Eads, for constructing South Pass Jetties.

Channel to be Obtained.	Cash Payments.	Annual Payments for 20 Years.	Withheld for 10 Years, U. S. Paying Interest.		Withheld for 20 Years, U. S. Paying Interest.	
			Principal.	An. Int.	Principal.	An. Int.
	$750 000					
25 x 200.....	500 000					
26 x 200.....	500 000					
30 (no width).....	500 000	$100 000	$500 000	$25 000	$500 000	$25 000
Total.....	$2 250 000	$2 000 000	$500 000	$250 000	$500 000	$500 000

Grand total.....	$6 000 000
Previously earned or advanced.....	2 000 000
Total involved in contract as amended.....	$8 000 000

Any one interested in verifying the accuracy of these tables, can do so by consulting the River and Harbor Acts of the dates named.

By comparing the two tables, it will be seen at once that the amended contract limits the depth of the navigable channel to 26 feet, and even reduces the width corresponding in the original contract from 300 feet to 200 feet. As I stated before, it is impossible to secure a minimum depth of 26 feet over a width of 200 feet without a greater central depth, and I estimated the probable central depth of such a channel at 30 feet. This Mr. Eads denies. I will not argue the question, but leave it to the judgment of the Members of the Society.

When Mr. Eads took his original contract, it was distinctly stated in the public press, and in the halls of Congress, that he was willing to risk everything, and only expected to receive pay as he accomplished certain definitely stated results. It now appears, however, from the statements contained in his last paper, that this was not seriously intended, and that he had an understanding with influential Members of Congress that he need not bother about the terms of the contract, as the latter could and would be modified. This is an extraordinary admission, and involves questions of political morality that I would prefer not to discuss. It is clear from results that Mr. Eads' confidence was not misplaced.

When I spoke of the large "bonus" that Mr. Eads had received over the cost of building the South Pass Jetties, I simply meant that all the money which he received over and above what he had earned under his original contract was really a bonus from Congress, as the latter

body was under no obligation to give him a dime that was not due under his "no cure, no pay" contract.

Even to this day, notwithstanding the extravagant claims made for the South Pass Jetties, the 26 by 300-foot channel of the original contract has not even once been obtained. To avoid any mistake, and to give the work the benefit of the latest developments, I wrote to the inspecting officer to get the latest facts, and he answers, under date of April 1st, 1886, that Mr. Eads has never had a channel 26 feet deep by 300 feet wide.

If, therefore, the original conditions had not been abandoned, and new conditions substituted, Mr. Eads would thus far have been entitled to but $1 750 000; whereas, I find, from the last Annual Report of the Secretary of War, that up to September 10th, 1885, Mr. Eads had received $5 150 000.

It must be stated, however, that he asserts that he could have obtained the guaranteed channel, but that Congress thought it unadvisable, and modified his contract. The inference is that the contractor was passive, and was only prevented from carrying out his contract by the interference of higher authority. Those who, like myself, were present during a portion of the session of the Congress in question, and were cognizant of the herculean efforts of Mr. Eads to save himself by a change of contract (without change of pay), cannot but smile at the ingenuity which throws the whole responsibility upon Congress. In view of this assertion, it is worth while to revive the memory of the method used to secure the desired change. No effort was made to have the items inserted in the River and Harbor Bill prior to its passage by the House of Representatives; but as soon as the bill reached the Senate, the paragraphs relating to the change of contract were inserted as "amendments." The bill as passed by the Senate being different from the bill as it came up from the House of Representatives, and the latter body having refused to accede to the Senate amendments, the amended bill was referred, as usual, to a Committee of Conference, consisting of three members from each body. This committee agreed upon a compromise in which the South Pass items were retained, and the compromise bill was then reported back to each House, and passed. In the House of Representatives no debate was allowed, and Members were compelled to accept or reject the bill as an entirety. As a consequence, the representatives had no opportunity whatever to discuss a change in contract by which the Government assumed new obligations amounting to millions. The eager anxiety and "great unanimity" of Congress in changing Mr. Eads' contract, can be judged from the above statement of how it was done. I ought to add that I do not blame the contractor for trying to save himself by securing a change in his contract, and that I cannot see why he does not frankly admit that he did his best to attain so essential a point.

It may be worth while, in this connection, to estimate the probable profit of the South Pass Jetties. This is a delicate subject, from which the light has been carefully excluded, but for many reasons it seems advisable and proper to enter on this inquiry. I know from the contractors' own books that the engineering expenditures (including the dredge boat) amounted on the 1st of January, 1879, to a little less than one and a half millions. The work which he then reported as necessary for the completion of his contract, he estimated at $362 111. Allowing liberally for subsequent expenditures, all of which were comparatively light, I conclude that the total cost of the works does not exceed two and a half millions. This gives the contractor a profit of nearly four millions; in this I include the one million in the hands of the Government, which has been earned but is retained as security, and on which the Government pays an annual interest of $50 000. After such an exhibit, it is not a matter of surprise that Mr. Eads is eager for another contract.

I regret now to have to call attention to a garbled quotation from my first paper, by means of which I am charged with making an incorrec statement as to the depth of the channel between the jetties. I wrote, "the engineer who built them contracted with Congress to give a depth of 30 feet *for a width of* 350 *feet* * * * and failed to secure such a depth." Mr. Eads omits the words that I have italicized, thus changing the meaning of my sentence, and then says that my statement is "incorrect." I repeat emphatically that my statement, as I wrote it, is absolutely true, and Mr. Eads himself nowhere denies it. Wherever he speaks of 30 feet depth, he means 30 feet "regardless of width," and, as I stated before, this central depth is the depth that in fact, and necessarily, goes with a 26-foot channel of the contract width.

The severe reflections on the last Board of Engineers on the South Pass Jetties are uncalled for. Mr. Eads claims that the Board was inconsistent in recommending that he be allowed to complete his contract notwithstanding their expression of opinion that he would be unable to secure a channel of the guaranteed depth and width. I can see no inconsistency in this. The contractor vigorously asserted his ability to carry out his full contract, and, while the Board did not believe that he could succeed, they thought it but right to give him the opportunity to prove himself correct and the Board in error.

It is evident from the contractor's subsequent course, that he had but little faith in his own statements, for instead of attempting to do what he was bound by contract to do, what he declared he could easily do, and what the Board wanted him to do (if he could), he at once proceeded to secure his release from these conditions. He now claims that his horse was "harnessed," but it is evident that he had no faith in his own skill as a driver.

Having thus fully substantiated all my previous statements about the South Pass Jetties, I will add some brief comments on other points of Mr. Eads' elaborate production. In this connection, I cannot forbear commenting on the tone of infallibility which pervades his whole paper, and on the manner in which he pours contempt upon all who have the temerity to disagree with him. It is his habit to write as if all army engineers were opposed to the construction of jetties at the mouth of the Mississippi; it may, therefore, be worth while to revive a few facts. In 1856, on the recommendation of a Board consisting of Commodore Latimer, of the navy, and Majors Chase, Beauregard and Barnard, of the army engineers, contracts were made for jetties at South West Pass and Pass à l'Outre, and under these contracts light jetties were built at both passes. Being merely piles, without stone or brush, they were soon destroyed, but not before they had shown a deepening of the channel between them. The amount appropriated ($330 000) was insufficient to secure substantial work.

The Board of Engineers convened in 1873 to discuss the improvement of the mouth of the Mississippi, reported in favor of a canal, except General Barnard, the President of the Board, and former member of the Board of 1856. He submitted a powerful minority report, advocating the trial of jetties at the South Pass. After much discussion, the whole subject was committed to a new Board, on which were three army engineers (Wright, Alexander and Comstock), one coast survey engineer (Mitchell), and three civil engineers (Sickles, Roberts and Whitcomb). This Board reported in favor of jetties at the South Pass, as Barnard had recommended, though General Wright expressed his preference for a canal.

It will thus be seen that of the army engineers who belonged to the two Boards last named, six reported in favor of a canal and three in favor of jetties, one of the three (Gen. Barnard) being, in Mr. Eads' estimate (in which I concur), the ablest civil engineer in the army. In fact Gen. Barnard is really the father of the South Pass Jetties, having selected the Pass and the methods of improvement, whereas Mr. Eads wished to take the Southwest Pass. To do full justice to an adversary, I must confess that I agree with Mr. Eads that the Southwest Pass should have been chosen. Congress, however, followed Gen. Barnard, and selected the South Pass, notwithstanding Mr. Eads' objections.

The truth is, opinion in the Engineer Corps was divided on the probable results of jetties at the mouth of the Mississippi, and the same division extended among civil engineers. While the then Chief of Engineers was decidedly opposed to jetties, many of his juniors did not agree with him, and some of them, as shown before, reported officially in their favor, notwithstanding the Chief's opposition. The statement, which is frequently made, that the Corps of Engineers, as a body, were

opposed to the construction of jetties at the South Pass is not warranted by the facts.

I will not discuss the subject of Galveston Harbor, as I know nothing about it, except from reports. My object in writing the discussion of Mr. Corthell's paper was to give Col. Mansfield, who is not a Member of the Society, an opportunity of replying to Mr. Corthell through me, so as to place his reply before the Society that printed the attack.

I notice toward the latter part of Mr. Eads' paper, a statement that Mr. Corthell has shown by a letter from Mr. Forrest (Secretary Inst. C. E.) that military engineers in England are not employed on hydraulic works. This sentence is an ambiguous one. If it means that Officers of the Royal Engineers do not do hydraulic work in England itself the statement is probably correct, but it is no answer to what I stated. I said "Great Britain makes exactly the same use of her military engineers as does the United States." This is true, and it can readily be proved. The civil work of the Royal Engineers is done in the colonies, and especially in India. They planned and built the great Ganges Canal; all of the Indian railroads; many large bridges, dams, tanks, and systems of irrigation; made the Geodetic Survey of India; and they are now employed on all classes of engineering work. In writing to this Society, I need only refer to the "Roorkee Treatise on Civil Engineering" (written by a Royal Engineer), to the Thomason College of Civil Engineering (under charge of a Royal Engineer), and to Major Cunningham's "Hydraulic Experiments."

In the *Journal of the Royal United Service Institution*, is a paper by Captain Hoskyns on the Afghan Campaigns of 1879–81, where 117 Engineer-Officers were employed, in which the author states that the officers who excelled "were those who, in peace time, had had opportunities of organizing labor and of exercising their own resources." Colonel St. John, in discussing this paper, remarks, in regard to Engineer-Officers, "I can testify from my own experience that the best men on service are those who were employed on the great railway and canal works of India." General Maclagan's testimony is much stronger on the same subject; and General Sir James Hills corroborated Colonel St. John, as did the chairman, General Sir J. Linton Simmons. Such statements, from officers of high rank, are a conclusive answer to the assertion, often made, that civil work unfits an engineer for military service. In this connection, I am reminded that the late General Weitzel told me that, on a visit to Germany, he was asked by German Engineer-Officers as to how American Engineer-Officers were employed in time of peace; and when he stated that they were placed in charge of river and harbor works, the German engineers expressed their hearty approval of the action of the American Government, and hoped their own might be induced to follow the American example.

Since writing the above, I have had an opportunity of examining the

latest statement of stations and duties of the Royal Engineers—that for March 1st, 1886—with the following result:

I find a very large number of officers on survey duty all over the world, notably in Great Britain, Ireland, India, Cyprus and Africa; some in charge of or on duty with railways in India, including surveys for new railways; many on duty in India in charge of the Ganges Canal and other systems of irrigation, including tanks; and several similarly employed in Egypt; a large number on duty with telegraph lines in India, Persia and elsewhere; many as principals or professors at the Civil Engineering Colleges at Cooper's Hill, Madras and Roorkee; some in charge of works under the Admiralty and constructing dock-yards, as at Portsmouth; a large number engaged in the Public Works Department, the nature of whose duties is not specified; several Directors of Roads (presumed to be highways); one Conservator of Forests; one Engineer-Architect; one Curator of Historical Buildings, etc., etc.

Any amount of additional evidence will be forthcoming, if desired, but I imagine that the above will suffice to prove my point. It might be pertinent to add, in conclusion, that Major Cunningham's "Hydraulic Experiments" is one of the publications of the Institution of Civil Engineers, of which Mr. Forrest is the Secretary. The latter doubtless meant that the Royal Engineers do not do civil work in England itself.

Mr. Eads' paper concludes with a disparaging allusion to "tentative" engineering, as if there were no problems in engineering that could not be solved immediately. I will only remark on this point that the older and wiser one grows, the more he realizes the many-sidedness of all questions, and the less he is inclined to make positive predictions. His references to army engineers as "military students" are in very poor taste, in view of the fact that all of the "students" of his own age have had at least twice as much engineering experience as himself. For my part, I hope to remain a student to the end of my days, and I have no desire to be classed among those whose education is already finished. The Corps of Engineers, while making no claim to infallibility, is not afraid to point to its river and harbor work as the best answer to the slurs contained in Mr. Eads' paper.

E. L. Corthell, M. Am. Soc., C. E.—The great length which the discussion of this paper has already reached, the numerous statements of facts, and the variety of opinions expressed, make it unnecessary to extend the discussion further, for fear of repetition. The replies to Colonel Merrill, first by the author and then by other members, especially by Mr. Eads, cover the whole ground so completely, that it is only necessary now to take up a few salient points that have been presented by other members who have criticised the author's paper.

Mr. McMath bases quite a long paper on the assumption that the conditions existing at the jetties are not applicable to the upper reaches of the river; and that, therefore, some of the lessons which the author gave are not correct in their application to other portions of the river.

The author will be obliged to dissent from his assumption, and to repeat that the conditions found in the upper river, or below Cairo, do exist at and near the mouth of the river, except that the slope of surface at flood stages is considerably greater in the upper than in the lower reaches of the river. The velocity of the current at New Orleans, and even below there, often reaches five to six miles per hour. The irregularities of the bed and the crooked course of the river, and other conditions, produce the same disasters that occur above. It is only necessary to mention Bonnet Carré, Morganza and Davis' Crevasses; also the jump, twelve miles above the head of the passes, and Cubit's Crevasse, or Gap, three miles only above the head of the passes, to show that the river in these lower reaches is no aged dotard, enfeebled by the loss of energy, as Mr. McMath would make him out to be. From New Orleans to the mouth of the passes there are scars all along the banks, where the river has in years past inflicted deep and gaping wounds which it has required many years to heal.

The author ventures the assertion that there are conditions existing in the delta of the river that far more complicate the improvement of the channel than those which exist in the upper river. The inflow of sea water under the fresh water; the peculiar conditions existing at the head of the passes, where the tide, salt water, fresh water and sediment, in varying velocities and quantities, presented conditions which made the problem difficult, and compelled the handling of a million cubic feet of water per second; and that, too, where the banks and the bed of the river and passes are so easily abraded, that the greatest possible care had to be taken to prevent serious mishaps by rapid enlargement of channels which it was important to preserve unchanged.

The whole problem there is a working model on a very large scale of the conditions, principles, works and results as applied to the upper river. First, two bars removed and the channel through them deepened, in one instance from 15 to 40 feet, and in the other from 9 to 31 feet. Second, wide places narrowed, and the river brought to a uniform width by "high water treatment." Third, outlets closed, as at Grand Bayou, which previously carried off to the Gulf one-third of the volume

of the pass. Fourth, shutes closed, as that around Crane Island. Fifth, one of two main channels closed, as at the island at the head of the passes, thereby reducing the friction and lowering the surface slope of the floods. Sixth, jetties, dykes, permeable and solid wing dams, protection of the bed, the use of willows loosely piled or in thin mats on edge, or in mattresses laid flatwise; deposits made, solid ground formed in the old channels with trees now growing upon them, and the works protected by these deposits and the main channel made permanent.

The lessons should, therefore, be heeded ; if they are not, the attempt to improve the Mississippi River will, in our opinion, prove an utter failure.

The remarks of Captain Bixby show such a meager knowledge of the principal subjects under discussion, namely, the South Pass Jetties and Galveston Harbor improvement, that it is unnecessary to take much time in answering his statements.

The expression, "the present channel (at South Pass Jetties) may fill up rapidly as soon as the present dredging stops," is a very remarkable one, since the official reports of the construction and maintenance of the jetties show that dredging did not only not make the channel, but also that the dredge boat which was used to pare off the irregular sides and bottom ridges, to bring the channel to the exact requirements of a Congressional enactment, has not been used at all since February, 1883.

In reference to what he criticises as the author's "sweeping condemnation" of the Galveston works, he states that they should not be judged in their present condition, for it would be like criticising a building when only the foundation and lower story were erected, and that the author has mistaken this part of the structure for the whole house. Had the first story, or even the foundation of the North Jetty been in sight or been found by a lead line, and if the first story of the South Jetty had not been completely swept away, and the foundation that remained sapped by the currents and its structure ruined by the *teredo*, there might be some point in his remark, and some injustice in the author's "sweeping condemnation."

And, third, a still more remarkable statement is made in connection with the subject of civil and military engineers. He states that "they (West Point graduates) have, prior to the late rapid development of the United States, individually planned and constructed almost all the great civil engineering works in the country." We

leave this statement with the Society after simply mentioning Ellicott, Canvass White, Samuel H. Kneass, De Witt Clinton, Jervis, Laurie, Kirkwood, Welch, Roberts, Roebling, McAlpine, Whipple and Francis, who, with others that might be mentioned, were the pioneers of engineering construction in this country, and who developed its resources by surveys, canals, railways, bridges, water-works and water-power, and the improvement of rivers and harbors.

General Gillmore takes exceptions to the author's remarks on his work at Charleston. There was no space in the original paper to enlarge upon the subject, or to explain. His discussion compels the author to do so now.

The point that the author made was in reference to submerged jetties, jetties not reaching deep water, and a very gradual process of construction, and the statement was made that "the neglect of these lessons (from South Pass Jetties), or rather the plans and constructions carried forward on opposite principles, have at Galveston, Texas, and Charleston, S. C., resulted in complete failure," and that there was two feet less water at the latter place than when the works were begun. This information was gathered from the Official Reports of the Chief of Engineers, and, at the time the paper was presented, there had been about *twelve* years of work done at Charleston, partly on one plan and partly on another. The first plan contemplated the removal of an old jetty which projected into the Beach Channel. The work was commenced in 1871 or 1872, with the expectation of obtaining by it a depth of 20 feet at mean low tide. (See Chief of Engineers' Report, 1874, Part II, page 5.) At about this time the City of Charleston, in 1874, had, by dredging slightly, obtained a good channel of 18 feet at high tide, or 13½ feet at low tide, in what is called the Pumpkin Hill Channel; and it was stated by the Army Engineer that *two feet more* was practicable by the same process; that is, 20½ feet at high tide and 15½ feet at low tide. Even in the Beach Channel, which was under improvement, there was, at the time, a depth of 14 feet at mean low water. And General Gillmore, in a letter dated January 3d, 1875, while this work was going on, stated, "I see no reason to apprehend that the channel would not remain permanent if dredged to a uniform low water depth of 16½ feet to 17 feet, corresponding to 21½ and 22 feet at ordinary high water. (See Chief of Engineers' Report, 1875–76, Part II, page 31.) But evidently the principles and con-

ditions were not understood, for the very work (the removal of Bowman's Jetty) which was intended to deepen the channel, actually shoaled it; for on page 31, Part I, 1876, Gen. Gillmore says: "Since the completion of the removal of the outer end of the jetty, that portion of the channel that was before deep has shoaled considerably;" and he recommended at that time the expenditure of $10 000 in the attempt to remove this advancing shoal by dredging, and did expend that sum without accomplishing any good result. At the close of the fiscal year, 1876, there had been expended $88 637.93, mostly in the removal of Bowman's Jetty, and partly in the removal of sunken wrecks. Later surveys demonstrated still more clearly that the plan was ill-advised. A survey made July 14th, 1877, led to the following statement: "It (the survey) showed quite conclusively that the shortening of the Bowman Jetty had not produced a wider channel at that point, but that the west end of Drunken Dick Shoal approached the Sullivan Island shore and the jetty as fast as the latter was reduced in length. Neither was the channel deepened by the shortening of the jetty; on the contrary, the greatest depths on the smallest cross-section were less than existed on the smallest cross-section four or five years ago." The failure of this plan led to the devising of another, contemplating submerged jetties, with gaps to be left at the *shore ends very low to allow the tide to flow in.* The Beach Channel Jetty was to be held 12 feet below low water and the main channel jetty 15 feet below low water, both jetties gradually reaching the surface of the water towards the end of the jetty, one 1 500 feet from the sea end, and the other 2 000 feet. One object also of the low jetty plan was to prevent the advance of the bar; and, in order probably to be sure that the right plans were adopted, the process of construction was to be "very gradual."

The end of these jetties, as laid down at that time, did not reach the 24-foot curve by 2 miles, and the 30-foot curve by 4 *miles*, as the slope seaward was very gradual in the direction of the jetty prolongation. The Army Board approved these plans. These officers were Generals Tower, Wright, Newton and Gillmore. The jetties have recently been extended beyond the original limits, perhaps to keep pace with the advance of the bar.

Without going into a detailed description or discussion, the author will simply state that the estimated cost was $1 800 000, and that up to June 30th, 1884, about the date of his paper, there had been

expended $1 044 768.25, or about 60 per cent. of the estimated total cost. The following are some of the results: The original depth of the channel around Bowman's Jetty was 14 feet in 1874; in 1881 the depth through the jetty channel was 13½ feet; in 1883, 11½ feet; in 1884, about 12 feet. But the area of shoal ground between the jetties has not decreased, for there was in 1881 about 4 000 000 square feet of shoal above the 12-foot curve; in 1883, 12 000 000 square feet above the same plane; in 1884 about 7 500-000 square feet. In 1874 the main ship channel, which has always been used by the deeper draught vessels, had a depth of 13½ feet; but the jetty works have caused a marked deterioration in this channel with no corresponding increase of depth in the jetty channel. The "very gradual" process of construction advised and carried out, has, in the opinion of the author, been more than anything else, the cause of the considerable advance of the bar, which, as near as the author can make out from the various charts, has been about 1 000 feet to 1 200 feet.

The exception taken by General Comstock to the author's remarks that "jetties, to be of the greatest effect, should be brought above the surface of the water at flood tide," and "they must be kept high to prevent the wash of sand over them into the channel," after the detailed description and history of the Galveston Jetties given by Mr. Eads, supplemented by the charts of Galveston Harbor (see maps accompanying paper of Mr. Eads), is shown to have been not well taken. The principles enunciated in the paper of Mr. Eads in reference to the reduction of friction by the reduction in width, also bears directly upon this point.

In reference to the advance of the bar off the South Pass Jetties, the author's statement was made from the only report available at the time, which was that of 1883. Comparing 1876 with 1883, there had been in the 1 000 feet width immediately in front of the jetties and in the sweep of the discharge, as near as could be ascertained from the small chart before us at the time, a shoaling of 1.6 feet over two-thirds of this area, and a deepening of 3.1 over one-third of the area, or an average shoaling of one-tenth of a foot over the whole area. During the years 1882–83 there had been an average deepening of 2.3 feet over this same area. An inspection of the table of changes that have taken place in scour and fill over all the sub-areas of the large fan-shaped area, will show that during eight years there have been marked annual irregu-

larities in the action, due to the varying conditions of winds, tides, littoral currents, river discharge, and quantity and character of the sedimentary discharge. The annual report of the inspecting officer of the Government for June, 1885, received since the author's paper was read, shows that over the *whole fan-shaped area* in front of the jetties, there has been an average *deepening* during the last year of 1.16 feet, and an average deepening directly in the line of the river's discharge of 2.5 feet. No one has ever, to the knowledge of the author, stated that the bar would not re-form in course of time; on the other hand, it was stated by Mr. Eads, and others supporting him, at the time of the discussion of the subject, that the bar would re-form; and also that the action of the sedimentary discharge would be as stated by General Comstock; that is, the strong current effected by the jetties would throw the sedimentary matter into deeper water than before; and any one can see that this would prolong the time of bar re-formation, which, without such extension of time, would be over 100 years. It must be recollected, also, that since the first works were built, and the material in the bar was moved by them into the Gulf, it is now a decade; that for nearly seven years the commerce of the world has found an unobstructed entrance, and that during every year that it remains, the jetties are paying for themselves many times over in reduced cost of transportation to the seaboard of the products of the Mississippi Valley, in insurance, and in regulating and holding down the rates of freight to the Atlantic ports by rail.

The effect of the jetties is, and will be, to deposit the sediment on the flanks of the strong issuing current, and to carry the residue to greater Gulf depths; thus accomplishing two advantageous results, namely, building up a foundation for future extensions when they may become necessary, and prolonging the time of the bar's re-formation in the track of the river's discharge and in the track of commerce.

A large part of the so-called "fan-shaped" area includes the incipient banks and shoals on the outer flanks of the jetties, and the changes upon it give no reliable facts bearing upon the advance of the bar.

As a matter of interest in closing this discussion, the following statements are given from the Annual Report of the Inspecting Officer, dated June 30th, 1885:

"On June 15th there was a least depth of 30.6 feet from the main

river into South Pass, with a least width of 100 feet for the 30-foot channel, and a very wide 26-foot channel.

"Throughout the length of South Pass there is a 26-foot channel, whose width varies from 270 feet to 900 feet. The least depth of the channel on sections one mile in length is from 29.2 feet to 46 feet.

"The least depth through the jetties is 31.3 feet; the least width of the 26-foot channel is 270 feet, and the 30-foot channel 130 feet.

"Beyond the ends of the jetties there is a central depth of 31.6 feet, the 26 and 30-foot channels being 220 and 80 feet wide respectively."

The author ventures the prophecy, that in less than two decades from this date, the wisdom of the jetty plan will be so fully proven by a long period of time, and the commerce of the Mississippi Valley will have grown to such enormous proportions by the constant development of the country; by the successful improvement of the Mississippi and its tributaries; and by the opening of an interoceanic communication with the Pacific countries, thus effecting the commercial extension of the river into the Pacific, that the Government will abandon the already inadequate channel into and through the little South Pass, and will construct jetties at the mouth of the Southwest Pass, where, from the main river to the Gulf, commerce will find a magnificent channel, through which a tow of half a dozen vessels can be taken, and set adrift in the deep water seaward of the jetties.

In reference to the best manner of conducting the public works of the Government, it was not necessary for General Comstock to go so far in finding an alternative plan to the present one. The plan proposed by him, all, of course, will acknowledge is faulty, and to be condemned as a system; and it is hardly fair to intimate that any such plan is in the minds of the civil engineers who are advocating a modification of the present plan of conducting public works. His alternative plan is certainly not in accord with the remarks made in the author's paper on this subject; for he stated that the Government should summon to its aid the best engineering talent, and that the doors should be thrown open to all engineers fitted by education and experience for the service required; and that the Government should not give the general or exclusive charge of public works into the hands of engineers educated to conduct works of a totally different character.

A more reasonable alternative plan, and one which would keep pace better with the wonderful development of our industries and the grow-

ing demands of commerce for deep rivers and harbors, would be the consolidation of all the civil works of the Government into a civil department, with its own Cabinet officer, to be called "The Department of Public Works," with the sub-departments of geodesy, geology, engineering (rivers and harbors), light-houses, architecture and weather service. The superintendents of each department to appoint their assistants from civil life, or, in times of peace, also from the army or navy by detail; sound and practical civil service principles and rules to prevail throughout the department; all connected with it to be selected, assigned and promoted according to ability, experience, faithfulness and length of service. The author ventures the assertion that such a system would be more attractive to the officers of the army and navy than the present one, and that many engineers in both arms of the service would gladly resign their commissions, if necessary, in order to hold permanent positions in this civil department. In place of several antagonistic and discordant elements seeking often to overreach, belittle and encroach upon each other, a complete and harmonious system would exist, which would utilize and draw to itself the best talent, the greatest skill and the largest experience of the various special professions that make up the great profession of engineering, which "seeks to convert, adapt and apply the great sources of power in nature to the use and convenience of man."

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(Vol. XV.—May, 1886.)

THE SPONGILLA IN MAIN PIPES.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

READ OCTOBER 15TH, 1884.

WITH DISCUSSION.

Great attention is paid by hydraulic engineers to the designing of dams, aqueducts, reservoirs, etc.—structures connected with the sources of supply; but the question may well be asked: Is proper attention given to the pipe or distribution system ?

Experiments and formulas we certainly have on the flow of water through pipes, but the writer is inclined to believe, as the result of observation, that when the water has been once turned into a pipe system, little more attention is paid to the condition of the pipes. Sometimes, it is true, small pipes fill up entirely, and then specimens are exhibited showing the growth of tuberculation. Again, the water becomes bad in one street while it is good in the adjoining neighborhood, and the

result is attributed to some mysterious agency. It is the belief of the writer that one of the great steps in advance that will be made in the years to come in the designing of pipe systems will be the introduction of facilities for cleaning out the pipes at stated intervals of time. This will be found necessary, not so much for the purpose of restoring the normal capacity of the pipes, as for maintaining the purity of the water.

A few years since, the writer was standing by the side of Prof. Remsen when he made the discovery of the *spongilla* as being the principal source of trouble in the Boston water.

Many excellent engineers have doubted the influence of the sponge, but in a somewhat varied experience in different sources since that time, the writer is inclined to give more and more weight to the conclusions of Prof. Remsen. Not to go into the question of the growth and development of the *spongilla* in lakes, reservoirs, etc., which may be made the subject of a future article, the writer would like to call the attention of the profession to the fact of the growth of the *spongilla lacustris* in the pipes of a water system. When the sponge is present in the sources of supply, pieces of it find their way into the pipes. These, decaying, give an offensive cucumber or fishy taste to the water. This accounts for the fact that the bad taste is almost always detected in the city a few days before the taste is found in the reservoirs. The sponge is really an animal and lays eggs, which float down with the water and attach themselves in large quantities to the interior surfaces of the pipes.

More accurately, these reticulated masses are the *statoblasts* of the sponge. The forms which these eggs take are very curious. They mass themselves in traceries like lace-work, generally from the size of a silver half-dollar to the size of a plate, and the exterior boundaries generally take a circular or oval form. Soon the eggs begin to bring forth the sponge, no matter what the pressure, and a soft velvety, light green sponge begins to grow in circular patches. This growth, if left undisturbed, gives out long fingers of sponge.

With life there must be death and decay, and with the decay of the sponge comes the cucumber taste.

The writer has seen large mains, under a pressure of 100 feet, where the entire surface, as far as examined, was filled with offensive masses of sponge closely packed between and around the tubercles. He has also seen them in all the stages of growth. Within a few days a break

in a 48-inch main gave an opportunity to examine the sponge in company with Prof. Hyatt, who recognized the young sponge as *spongilla lacustris*, var. *flexis pina*. Flushing will not remove this growth. Some form of scraper or wire brush is necessary. In Halifax, Mr. E. H. Keating, M. Am. Soc. C. E., has perfected and used extensively an apparatus for cleaning out pipes, but one of the difficulties of this form of scraper will probably be found to be the protrusion into the interior of the pipes of service-pipe connections, etc., which has been found to be necessary to keep rust from covering them. It would seem, however, that some form of brush might be devised which would do the work effectively.

DISCUSSION.

Joseph P. Davis, Vice-President Am. Soc. C. E.—The result of a good many experiments in Boston has been that, under the conditions which produce what is known as the cucumber taste, if the pipes are blown out the bad taste increases. The *spongilla* was discovered to be the cause of the cucumber taste some few years ago by Prof. Ira Remsen, of the Johns Hopkins University. There was a comparatively small amount of it in the pond where it was discovered; one could hardly believe such a small quantity could produce such a bad effect. The presence of a sponge-like growth in the Cochituate conduit was noticed soon after Cochituate water was introduced, but although in 1854 (I think that is the year) the cucumber taste rendered the water unfit for use, no one seems to have suspected that the taste arose from the *spongilla*.

E. B. Dorsey, M. Am. Soc. C. E.—Was that undoubtedly the cause of the bad water?

Mr. Davis.—There is, perhaps, some doubt about it. But we are inclined to believe that that was the cause. It is very curious how the presence of this taste is usually first made known. In 1876 it occurred something in this way. A consumer would come into the office and say the water supplied to his house was exceedingly bad. On asking him where he was now getting his water from, he would say, from his neighbor. "Was that good?" "Yes." Naturally we supposed a fish had got into his supply pipe. During the day, probably a number of persons would come in complaining of their water, whereas their

neighbor's water was good. In the next 24 hours the whole section would be troubled, and within two or three days it would be all over the city. In that year (1876) no bad taste was found in the reservoirs for two or three days after the discovery in the city. I do not know what the life of the animal is, but the effect produced upon the water by its decay lasts sometimes two or three months. The supply for Boston was (in 1876) brought to the city by a single aqueduct and delivered into two reservoirs, one of which, the Chestnut Hill reservoir, had two basins; the lower was quite a large one and as clean a one as I ever knew of. The first knowledge of any trouble in the city was had as I have described it. I was out some two or three days trying to find the source of the trouble, and found it in the lower basin of this reservoir. The supply was shut off for some three or four months, when it was again turned on. It did not seem possible there could have been any growth in the basin. Since that time Prof. Remsen has found this *spongilla* growing in the ponds.

J. J. R. Croes, M. Am. Soc. C. E.—In cases where this has grown in the pipes—the supply pipe—how long does it last? I suppose it has a growth and death, and decomposition comes on?

Mr. Davis.—The taste sometimes lasts for two or three weeks in the pipes. In the reservoir, where it was not carried away by the current, it lasted some two or three months in the winter.

Mr. Dorsey.—Does it recur at the same time of the year?

Mr. Davis.—I think not.

Theodore Cooper, M. Am. Soc. C. E.—Can you distinguish it with the eye?

Mr. Davis.—It attaches itself to the rocks and grows like sea-weed.

Mr. Dorsey.—Is it green?

Mr. Davis.—When fresh and alive.

J. C. Campbell, M. Am. Soc. C. E.—Has there ever been anything done to prevent it?

Mr. Davis.—When it first occurred a thorough blowing out of the pipes was made, but it resulted in an increase of the taste. Of late years, finding that this only made it worse, we did not do it.

Mr. Croes.—Has this ever occurred in a supply that is taken from water not exposed to light or air? If I am not mistaken it only occurs where the water is exposed to light and air.

Mr. Davis.—I don't think I ever heard of a case where the water was not exposed that it became bad from this cause.

Mr. Dorsey.—Have they ever tried the ordinary sand or gravel filter?

Mr. Davis.—In Boston? No, sir; not for the purpose of taking this taste out. Experiments have been made for other objects. Of course, it takes out little or nothing held in solution. When you get the taste you cannot filter it out. In the early days of the Cochituate supply, sponge filters on the house faucets were very generally used, but when the bad taste developed in 1854 they had no beneficial effect. Exposure of the water to the air for a short time, in small quantities, or boiling, usually destroys the taste.

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327.
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ON INCREASING THE ACCURACY OF A SYSTEM OF MAGNETIC BEARINGS OF A SURVEY.

By OLIN H. LANDRETH, M. Am. Soc. C. E.

READ NOVEMBER 14TH, 1884.

WITH DISCUSSION.

This system is applicable to a closed survey or to a traverse. It often becomes necessary to assign a set of magnetic bearings to a survey in which the horizontal angles have been measured with a transit or a theodolite. The required system of bearings must conform to two conditions:

First.—It must be the most accurate system attainable from the given data.

Second.—It must conform to the vernier system by giving angles between adjacent courses identical with the corresponding vernier angles.

These two imposed conditions are best satisfied by the following method of survey and reduction:

1st. Measure and record the traverse angle at each station referred to the first line of the survey as an origin.

2d. Observe and record the magnetic bearing of each line, taking as such bearing the average of the bearings taken at the two ends of the line.

3d. Apply the traverse angle, with its sign reversed at each station,

to the magnetic bearing of the line starting from that station, by so doing forming a series of bearings of the first or origin line, each transferred from a separate line of the survey.

4th. Take the arithmetical mean or average of the several values of the bearing of the first side as the adopted bearing of that side.

5th. Apply the traverse angles, with their true sign, in turn to the adopted bearing of the first side, giving, respectively, the corrected bearings of each side.

This last operation may be facilitated by inspection, remembering that the corrected bearing of each line differs from the observed bearing in amount and direction, the same as the adopted bearing of the first line differs from that computed bearing of the first line which comes from the line in question.

If traverse angles have not been measured, but deflection angles, construct the corresponding traverse angles for the several stations by adding all deflection angles to the right and subtracting from the sum all deflection angles to the left which occur between the first line and the line in question.

The magnetic bearings may now be transformed into astronomical bearings by applying the magnetic declination.

It will be observed that the final corrected bearing of each line is determined from the bearings of every line of the survey, and hence is more accurate, both as regards ordinary instrumental errors as well as the effect of local attraction, than the bearing from one or even many observations at each station.

The unknown effect of local attraction at the several stations of an extensive survey may be assumed to be free from any tendency to a common direction, and hence these effects can be treated as accidental errors, making the degree of accuracy proportional to the square root of the number of stations.

Should the observed bearings of the individual lines be deemed of unequal accuracy, relative weights may be assigned to them, based on the candid judgment of the surveyor, in which case each computed bearing of the origin side is to be multiplied by the weight of the observed bearing of the corresponding line, and the sum of these products for all the lines is to be divided by the sum of all the weights, to give the adopted mean value of the bearing of the origin side, or the general mean.

EXAMPLE.—CLOSED SURVEY.

STATION.	Deflection Angles.		Observed or Computed Traverse Angles.		Observed Bearings.		Computed Bearings of First Line F. A.		Corrected Bearings.	
	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.	Deg.	Min.
A.....	98	12	98	12	N. 46	30 W.	N. 46	30 W.	N. 46	32 W.
B.....	38	18	136	30	N. 51	45 E.	46	27	N. 51	40 E.
C.....	34	07	170	37	N. 90	00 E.	46	30	N. 89	58 E.
D.....	89	14	259	51	S. 56	00 E.	46	37	S. 55	55 E.
E.....	72	13	332	04	S. 33	15 W.	46	36	S. 33	19 W.
F.....	27	56	360	00	N. 74	30 W.	46	34	N. 74	28 W.
					N. 46	30 W.				
							Aver'ge 46	32+		

DISCUSSION.

RUDOLPH HERING, M. Am. Soc. C. E.—Mr. Landreth's system is, I think, a good way of distributing the bearings over a closed survey; but unless this survey be small, and the territory free from local attractions, I cannot see that the magnetic bearings, particularly the corrected ones, could be of much use after a complete vernier circuit has been made. To correct the error of local attractions, and alter the magnetic bearing at any one point, would often be to mislead any one thereafter trying to retrace a course on the ground.

It is better, I think, to record the actual magnetic variation, even if the magnetic circuit does not close (and the date, inasmuch as it varies periodically), and not to try to average it for a whole circuit. On the survey I am at present conducting, the method proposed would be of no value, as we find numerous local attractions of great amount, owing to the geological character of the ground.

J. B. DAVIS, Jun. Am. Soc. C. E.—Mr. Landreth's method of assigning magnetic bearing to the courses of a survey, is the one I should certainly practice if there were no local attractions involved, if I had occasion to do such a thing. In my own experience I have never had such occasion; neither do I find myself able to conceive of such an occasion. We are discarding all needle work in Michigan as fast as possible. I need not say why. The least experience with the needle will furnish plenty of reasons.

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MEXICAN BRIDGE CONSTRUCTION.

By J. FOSTER FLAGG, M. Am. Soc. C. E.

READ DECEMBER 17TH, 1884.

The accompanying sketch is submitted as being rather remarkable for the work, from his own design, of an ordinary uneducated Mexican laborer, or *peon*—combining as it does, crudely, several principles of bridge construction.

Bridges in Mexico are, generally speaking, built of arched masonry, anything like a truss, especially in the section of country where this bridge was built (the State of Colima), being before the advent of railroads almost unknown. In the State of Colima there are but few bridges of any description—the streams being crossed, when possible, by the primitive method of fording—and these few are the usual arched structures. The River Armeria, crossed by the bridge sketched, is, for a long distance above and below the bridge site, too rapid (having an average slope for miles of one per cent.), and generally, even in the lowest stage of water, too deep to be fordable. And the size of the river in flood (then 800 feet wide and 25 feet deep in the channel), and the instability

of its stony bed, make it altogether too expensive a matter with the limited means of the country to build a permanent high bridge. It was attempted some thirty-six years ago, thirty miles farther down, where the current is much less rapid—a structure of the usual character (a series of brick arches of limited span) of probably 800 to 1 000 feet in length being then built; but a freshet, the following year, cleaned out the whole structure, except an arch or two at one end.

The *peon* referred to was occupied some four years ago as a ferryman, where the trail for cargo mules crosses the river, carrying across the mule packs, pack saddles, etc., in a "dug out." And if any animals could not be forced to swim the rapid current by pelting them from the banks, he stripped himself, and, seizing the bell mare or riding animal by the mane, swam beside her, and forced her across to lead the rest. About that time he happened to see a *Harper's Weekly* (probably sent to some one of our engineers) which had in it an illustration of a suspension bridge. This was a new light to him, and he revolved the matter over in his mind to see if he could not imitate the bridge in the materials at his command, viz.: the round sticks and vines cut from the forest, and small rough-hewn sticks of timber. As a result he put up a structure closely imitating the ordinary suspension bridge, the cables and suspenders being twisted from wild vines (*bejucos*), and the former passed over rude frames for towers, and anchored to huge boulders in the river banks. It was built without any nails or iron of any kind. It was, of course, a frail structure, but it served very well for foot passengers, and for carrying across, on wheelbarrows or the backs of *peons*, the cargo of the mules. The writer found it quite an assistance in passing backwards and forwards men and tools employed in building a railroad bridge at the same site.

But a heavy freshet occurred the same year the bridge was built and destroyed every vestige of it. Finding it profitable, the *peon* engineer decided to renew it; and this time he was not satisfied with copying another's design, but originated the one submitted with this paper. Like the first one, it was put together without nails or metal of any description—the suspension cable, as before, being made of wild vines twisted, and all joints tied together with lighter vines used when green, no manufactured ropes even being used.

The piers were made by driving light poles a short distance into the river bed, in the form of a square, tying them together with other poles,

SKETCH

OF A

MEXICAN BRIDGE

BUILT BY A COMMON PEON

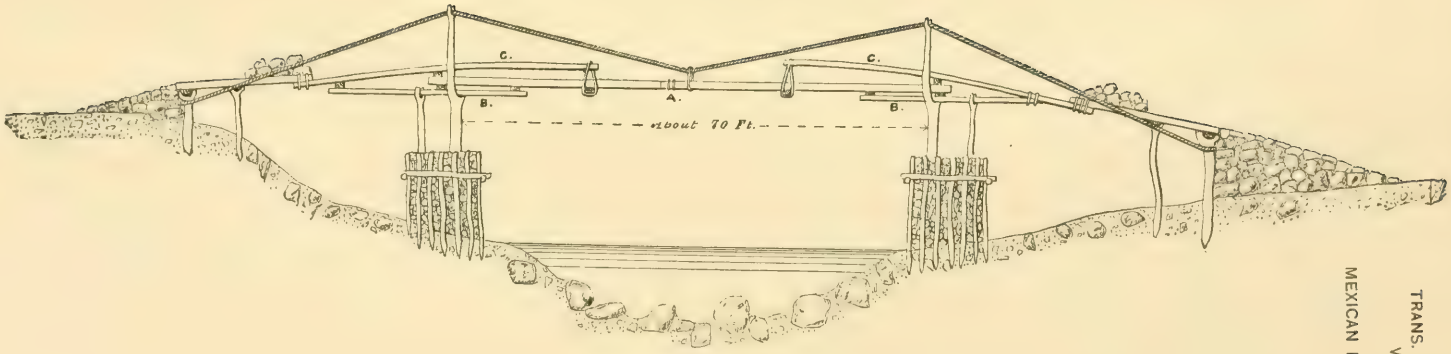

PLATE XIV.
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MEXICAN BRIDGE CONSTRUCTION.

and filling with stone. The stringers of the main span, in two pieces, were tied together with *bejucos* at *A*, and the spliced stick supported near the joint by the suspension cable—the only use to which the cable was put. The towers were natural forked sticks; forked to support the cable, and forked to support the corbels (*B B*) which assisted in shortening the main and lateral spans. And finally the long stringers were supported again, midway between the end of the corbels and the cable attachment in the center, by crude cantilevers (*C C*) which were loaded with stone near their shore ends to balance the weight of the central span.

The roadway, of rude joists and boards, is not shown. It was quite narrow, only one animal being able to pass on it at a time. The bridge proved to be strong and rigid enough to pass mounted men or loaded mules, and so served better than his first construction. It was in use, I believe without any accident, for some 18 months, when another heavy freshet unfortunately sent it the way of its predecessor.

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EXCAVATION AND EMBANKMENT BY WATER POWER.

By EDWARD BATES DORSEY, M. Am. Soc. C. E.
READ DECEMBER 17TH, 1884.

I wish to call the attention of the Society to a plan by which large excavations and embankments can be cheaply made—which is especially applicable to earthen dams—by simply applying the method used in the hydraulic mines that have been so largely developed in California. The system, in brief, is discharging the water under a vertical head of from 100 to 300 feet against the bank to be excavated. The momentum of the water cuts the bank, the material of which is conveyed by it into the flume, and thence by it to the place at which it is to be discharged.

This point of discharge is generally in some water-course or river, which soon becomes dammed by a perfectly water-tight dam, which remains intact and tight until it is destroyed, or partially so, by the winter's flood washing away the material from the top.

The water used in the mines is generally brought from reservoirs in the mountains. The dams forming these are brilliant examples of bold and cheap construction; among the principal of these is the Bowman Dam, on the headwaters of the Yuba River in Nevada County, California,

built under the general direction of Mr. Hamilton Smith, Jr., M. Am. Soc. C. E. Its maximum height is 100 feet, and length 425 feet.

The water from the reservoirs is conveyed through ditches, tunnels, flumes and pipes to the mines; in some cases aggregating a length of several hundred miles. These flumes show at times very bold engineering, being hung by iron rods to the vertical sides of the mountains; at other times they cross wide and deep valleys upon high and long trestle-work. When the valleys are too deep to be crossed in this manner, inverted siphons are used, made of wrought-iron sheets riveted into pipes. The great pressure these pipes stand, their lightness, durability and cheapness, will, in the future, force their adoption in many cases upon the hydraulic engineer. Some of these works, gener ally known as mining ditches, eclipse in magnitude and boldness the water-works supplying most of our or European large cities.

The unit of measurement of water in California is what is known as the miner's inch, which varies in different localities; the inch that is generally adopted equals 17 000 U. S. gallons in twenty-four hours.

The quantity of earth that this miner's inch will remove varies very much, being from 1½ to 9 cubic yards daily. Perhaps the most correct data is from the workings of the North Bloomfield Gravel Company, under the direction of Mr. Hamilton Smith, Jr.

During 1874–75, one inch of water removed 4.80 cubic yards gravel.
" 1875–76 " " " 4.17 " " "

In many cases the engineer in constructing dams could do better than this, as he could pick out the soft places and leave the hard, and not waste time and water in cleaning the bed rock or working the hard blue gravel which is generally richest in gold, and in mining operations must necessarily be mined and removed, even if it is done slowly.

In order to be perfectly safe, I have estimated that the miner's inch of 17 000 U. S. gallons daily will remove 3½ cubic yards. Upon this basis, actual bids were received for the construction of the plant, embracing engines, boilers, etc., to pump 20 000 000 gallons 200 feet high in twenty-fours hours.

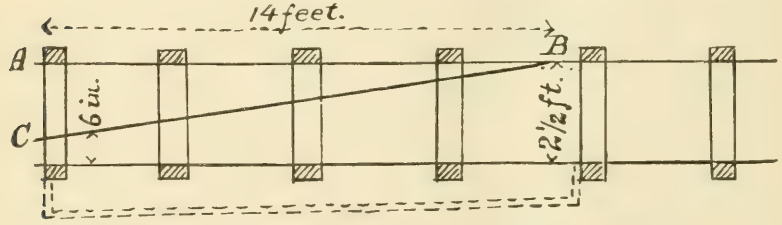
Recently I had to estimate the cost of constructing a dam 80 feet high. The surroundings prevented the use of masonry; the nearest material that would answer for puddle would have to be hauled four miles up a very steep grade; the only material available for building the

body of the dam was a sandy gravel extending to the greatest explored depth. By numerous tests the voids ranged from 8 to 23 per cent., averaging 10 per cent.

As the building material, as well as the foundation, was bad, I deemed it prudent to give the dam extra width; consequently the top was estimated 200 feet wide, with slope 2½ to 1 on each side, making the bottom 600 feet. One of the reasons for giving this great width was to get plenty of fine material for the interior filling that would take the place of the ordinary puddle. The calculated quantity of material was 1 757-127 cubic yards.

The plan proposed was to pump the water to an average height of 200 feet; of this, 120 feet would be available for hydraulic purposes, the remaining 80 feet would be lost in friction, fall of the flume, and dump. It was proposed to convey the water from the pumps to the banks by the usual hydraulic mining-pipe, made by riveting together sheets of wrought-iron. The water and the material would be conveyed to the proposed dam embankment in a flume 2.5 x 2.5 feet, made out of 1½-inch planks, with a fall varying from 5 per cent. at the beginning of the work to 2 at the close. This flume would be made similar to the ordinary mining flume, except that it would not be paved in the usual manner, either with stone or wooden blocks, but the bottom would be lined with thin sheets of steel plates, to prevent the wearing of the planks and to make the friction of the water and the conveyed material as small as possible. A few feet from the discharging end of the flume the upper portion of the side of the flume towards the center of the embankment would be cut away, commencing, say at a point 14 feet back where the height of the side would be 2½ feet, and diminishing to, say, 6 inches, at the end, thus:

SIDE ELEVATION OF FLUME.



A B represents the outside of flume 2½ feet high.

C B represents the side towards the center of the dam as cut away.

The dotted lines represent a trough or box to catch the overflow from *B C*.

By this plan, most of the water containing the lighter and finer materials only, would overflow into the trough represented by the dotted lines as above, and be conveyed by smaller branch flumes towards the center of the embankment, where it would be discharged and settle, forming a center of fine puddle; the stones, gravel and coarse sand would be on or near the bottom, below the side opening, consequently they would be discharged at the end, forming an excellent rip-rap for the embankment. This side opening could be raised or lowered as the material or work might require.

I will briefly describe the plan that I proposed to adopt in building this dam, using for example a dam 80 feet high and 200 feet wide on top, with slopes of 2½ horizontal to 1 vertical.

FIG. 1.

Fig. 1 represents the ground prepared for construction. *A* the usual puddle ditch, the material from which would be used to make *B* and *C*, small embankments, ten to fifteen feet high at the extreme base of the main embankment, this being to retain the muddy water until the finer particles held in suspension are deposited, so as to waste as little material as possible.

FIG. 2.

Fig. 2 represents the first hydraulic working, the flumes *F F* being 20 feet above the bottom—one on each side of the embankment. So that there may be no loss of time, while one is working the other can be moved or altered as desired, care being taken in this, as in all subsequent operations, to keep the outside or rip-rap portion of the wall closely built up to the discharging end of the flume. By doing this, a few planks, brush and stones can easily control and divert the water wherever desired.

The outer wall in this, as well as in all subsequent stages, should be kept as high as possible, in order to save the muddy water until the sediment in suspension is deposited.

FIG. 3.

Figure 3 represents the second hydraulic stage. In this the flumes *F F* are 40 feet above the bottom.

The upper part of the water in the flume, containing the finer materials, escapes over the low side opening into the trough and branch flumes *H H*, and is discharged over the puddle pit.

In this case it was estimated that one-fourth of the total quantity would be deposited here, and would make a most excellent puddling material. However, this proportion could be easily varied by raising or lowering the side discharge, as may be found desirable, to make the puddle finer or coarser.

On the figures the puddling material is represented by the broken horizontal lines, which also indicate approximately the form it would assume from the deposits made during the different stages.

FIG. 4.

Figure 4 represents the third hydraulic stage, the flume being 60 feet above the bottom of the embankment.

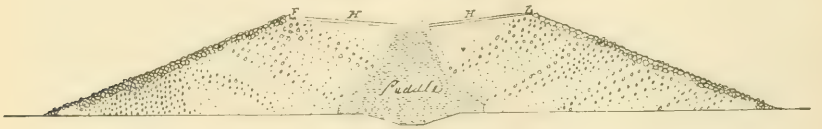


FIG. 5.

Figure 5 represents the fourth or last hydraulic stage.

In this case there was not enough water running in the stream to give an average of 20 000 000 gallons daily. It was proposed to keep the

water between the two embankments as long as possible, then to allow it to escape into the reservoir being formed by the dam, where it would deposit on the sides and bottom (which might be very desirable in some formations) the remaining sediment in suspension; after this it could be pumped again against the bank. By thus using the same water over repeatedly, very little fresh water would be required.

Cost.

The following table is taken from the able paper on "Hydraulic Mining in California," by Aug. J. Bowie, M. Am. Soc. C. E.*

Name of Mine.	Year.	Cost Per Cubic Yard Moved.		
		Water.	Total.	Total except Water.
French Hill Claim	1874-76	$0.014	$0.063	$0.049
Light Claim	1875-76	.006	.038	.032
Chesnan Claim	1874-76	.008	055	.047
Johnson Claim	1875	.006	.037	.031
Sicard Claim	1874-75	.004	.039	.035
North Bloomfield Gravel Mining Company	1874-75	.0077	.0207	.0130
Ditto ditto	1875-76	.0074	.0245	.0171

The two last were made under the superintendence of Mr. Hamilton Smith, Jr., M. Am. Soc. C. E., in his usual thorough manner, and give the correct cost, under favorable circumstances of high banks, plenty of water, good grade, and first-class management.

The engineer in any ordinary work could not expect to equal this work of Mr. Smith; but he would, in my judgment, except under very unfavorable circumstances, be perfectly safe in estimating at four cents per cubic yard the cost of digging, transporting, depositing the material in embankment, and all other expenses, except that of water and plant.

The preceding statement of expenses at the mines included many items of expense not necessary in ordinary engineering work, such as

* Hydraulic Mining in California. By Aug. J. Bowie. Transactions of the American Institute of Mining Engineers, Vol. VI, page 27.

loss of quicksilver, loss of labor and water in cleaning bed rock; watchmen to prevent robbery of gold; expenses of saving the gold; expenses of cleaning up, etc.

Cost of Plant.

A plant capable of pumping 20 000 000 gallons of water 200 feet high daily, which should remove 4 117 cubic yards of average earth or gravel, will cost, all complete and in working order, about $50 000, unless the local freight should be excessive. This is made from actual bids.

In this latitude this system could be worked all the year, except during the severe winter months, say for 200 days, in which time it would remove $4\ 117 \times 200 = 823\ 400$ cubic yards, so that the cost of the plant would be as follows per cubic yard removed. In work lasting

One year	$0.06
Two years	0.03
Three years	0.02
Four years	0.015

In this estimate nothing is allowed for the value of the plant at the end of the work. The engine, boilers, pumps, tools, electric lights, etc., would certainly bring something, which would reduce it materially.

In warmer climates, where the work could be prosecuted all the year, the above estimate would be largely reduced.

Cost of Water.

Wherever this system could be used in ordinary engineering work, the time that would be required would generally be too short to justify the purchase of compound or condensing engines, consequently the cost of pumping is calculated on the ordinary high-pressure engine consuming 36 tons of coal to pump 20 000 000 gallons daily 200 feet high:

36 tons of coal, at $4.50	$162.00
1 engineer	4.00
2 assistant engineers	6.00
2 firemen	4.00
4 laborers	6.00
Oil, waste, etc.	8.00
Total	$190.00

$190 ÷ 4 117: $.046. Average cost per cubic yard, say $0.045.

This estimate is based upon the supposition of pumping all the water 200 feet high, but in most localities a few cheap dams and ditches could be made to bring considerable water at an elevation that would materially reduce this pumping expense.

Resumé of Cost per Cubic Yard.

	Duration of Work.					
	1 Year.	2 Years.	3 Years.	4 Years.	5 Years.	6 Years.
Water.....	$0.045	$0.045	$0.045	$0.045	$0.045	$0.045
Plant.....	.06	.03	.02	.015	.012	.01
All other expenses.....	.04	.04	.04	.04	.04	.04
Total	$0.145	$0.115	$0.105	$0.100	$0.097	$0.095

In my opinion this plan will often enable the engineer to build cheap and safe dams where it would be impossible to build, at any reasonable cost, masonry or earth dams.

It could be used for excavating and removing all classes of earth or soil, except compact pipe clay. For constructing or building up embankments, it cannot be advantageously used where the earth for constructing the proposed work contains a great quantity of loam or clay; for this purpose the earth should contain a fair percentage of gravel and sand.

The cheapness with which the material can be moved by this system enables the engineer to use a much greater quantity than he would think of doing with the usual expensive systems.

Based upon my experience, I think a safe dam can be made of sand or on a sandy foundation, provided sufficient thickness is given to it, so that the head of the water will be neutralized by its friction in passing through the sand. This dam will undoubtedly leak, but probably not as much as required for compensation to the riparian owners on the stream below. This leakage will diminish with time, owing to the deposit of sediment on the bottom and sides of the reservoir. This deposit could be accelerated to any desired extent in the proposed plan by running the muddy water from the flume into the reservoir.

Wherever this plan can be adopted, it should commend itself to the engineer for several reasons :

First.—It will be much cheaper than the earth dam constructed in the usual manner—at least one-half where the material is good; and where the material is bad, the difference would be still greater, probably one-fourth.

Second.—It will permit the construction of earth dams where the material is so bad that the ordinary dam could not be constructed.

Third.—Owing to the cheapness, the dam could be made much stronger.

Fourth.—The dam will not settle or crack. It is as compact and as solid at the beginning as it can be made.

Fifth.—As it will not settle or crack, it is ready for use as soon as it is finished.

In the proposed plan, there is nothing new, except that the working head of 120 feet is derived from pumping machinery instead of gravity. This makes no difference in the result, except in increasing the cost. The prices of plant named in this paper are actual bids from manufacturers, with a guarantee not to burn more coal than estimated. In every other respect the plan is identical with that which has moved millions of tons in California, at a cost impossible by any other method.

DISCUSSION.

William R. Hutton, M. Am. Soc. C. E.—In the construction of the reservoir of Lac d'Oredon, in the Pyrenees (France), among other uses of water, the embankment was put in place by its means. A wooden box about 6 x 9 inches, with an inclination of 0.5 per 100, was placed alongside the railroad upon which the material was brought, and was extended as the work progressed. Two, sometimes three car-loads of earth (4.5 to 6.5 cubic yards) were dumped over the end of the box, and a current of water (about $\frac{1}{2}$ a cubic foot) was turned into it. The saturated earth was carried forward by the current, and down to the base of the embankment. Four or five men with hooks guided the large blocks of stone towards the faces of the bank. After reaching the bottom of the bank, the water flowed off, carrying only a little humus and some very fine sand. After a few minutes the material settled away from the water, and became very solid. Every space between the stones was compactly filled with sand, making a perfectly solid bank.

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(Vol. XV.—May, 1886.)

PERMANENT TRANSMITTING DYNAMOMETER.

By CHARLES A. SMITH, M. Am. Soc. C. E.

READ MAY 21ST, 1884.

In the fall of 1879, during the construction of the machine and engine-room of the Manual Training School at St. Louis, the author suggested that a permanent transmitting dynamometer should be made in connection with the use of the plant for experimental purposes, and the machine of which a rough sketch is given, was built.

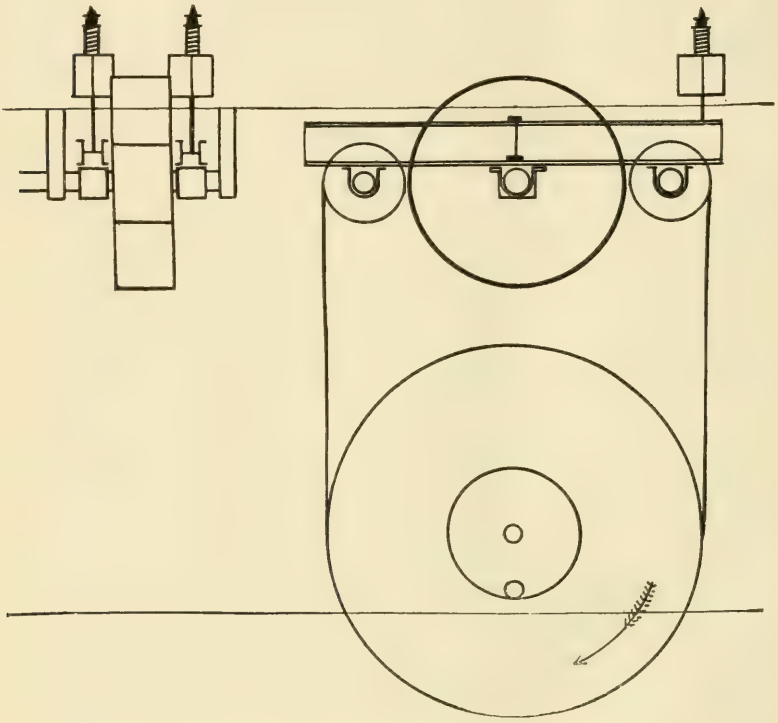
The type selected was one first used by Horatio Allen, Hon. M. Am. Soc. C. E., in the experiments with steam engines at the Novelty Iron-works, but with such modifications as seemed desirable. As it is now entering the seventh year of almost daily service, it is considered as having established itself.

The driving shaft of the engine is set vertically below the main line shaft of the building, and carries a driving pulley larger than that on the line shaft.

On each side of the driven pulley are two journals carrying a pair of balance beams which project beyond the driven pulley, and by means of journal and boxes carry two loose pulleys, one at each end.

These loose pulleys are so set that the belt from the driving wheel passes vertically to and from it under the driving and driven wheels, and over the loose pulleys.

The difference between the tension of the belt on the driving and driven sides is thus carried to the balance beams, which are kept from moving by a yoke attached to a spring resting on the floor above. The compression of this spring is used to measure the tension of the belt either directly or by the aid of a main integrating wheel and counter.



This form, with balance beam, is readily applied for permanent or temporary use. In the latter case, sheaves for rope would be advisable, as taking less room along the driven shaft, and the balance beams and driven pulley can easily be made of the width usually taken by a belt pulley.

The theory of the machine is so easily seen that it will not be intruded here. By taking the constants, in a convenient manner, the results are easily made very simple.

The vibration is about that of a friction brake, and is easily controlled.

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(Vol. XV.—May, 1886.)

DATA FOR FLATTENING THE ENDS OF RAIL-ROAD CURVES.

By ALBON P. MAN, Jr., M. Am. Soc. C. E.

READ NOVEMBER 19TH, 1884.

WITH DISCUSSION.

Assuming a curve uniform, except the flattened ends, as most scientific and convenient under ordinary circumstances, and that this flattening is only for the purpose of attaining proper elevation without intermediate improper elevation, throw of trains, etc., etc., the following is suggested:

1. That the ordinary rules for flattening curves are too seldom used, either because unsuited to the most important places, or requiring too much calculation or work. Rankine's rules give two different shifts to the same curve if the change at either end is not alike, as when to tangent at one end and reverse or compound at other, introducing thus a new complication.

2. That "Froude's curve" of deflections, as the cube of distances (or radii inversely proportionate to distance), applied to a curve shifted towards its center, is reasonably proper, suitable and convenient.

3. Since the uniform length of curve of adjustment here given is

200 feet (*i. e.*, 100 feet either side of *P. C.*, *P. C. C.*, or *P. R. C.**), it is assumed that all curves and tangents are at least 200 feet long, and that tangents of at least that length are left between reversed curvature (especially where the degree of curvature of the two curves added together exceeds 8 degrees), and this is always practicable, since an analysis of any alignment will show that such a tangent can always be attained by but a slight and unimportant variation in the line.

4. Although the data herewith perhaps give more than necessary length of flattening for light curves, yet the invariable length of 200 feet is chosen, upon the basis that what is suited to 6 or 8 degree curves at high speed will satisfy sharper curves, since speed is in practice reduced on such, and that what is practicable for sharp curves in the adjustment of location, will be easily attainable on light curves without material expense.

5. These curves are deemed adequate for a speed of say 40 miles per hour on 6 or 8 degree curves, and 60 miles on 3 or 4 degree curves; but it is hardly necessary to so flatten curves of more than a mile radius, or curves in yards or other places where trains are necessarily slow. The careful adjustment of these curve ends is considered as of most importance on trestles, bridges, or where the track is not easily readjusted after once fixed.

Explanations and Directions.

1. Shift all curves throughout their length radially towards their centers, each its degree of curvature multiplied by 0.2909 of a foot, as shown near bottom of Table I. These curves can be laid out, using the same deflections as for original curves, but a chain shortened from the usual hundred feet as shown immediately beneath the shift in same table. In such case, to save errors on account of difference of chaining, it is advisable to shift all the points, and chain the curves towards the instrument.

2. Lay off 100 feet along shifted line (or tangent) each way from original *P. C.*, *P. C. C.*, or *P. R. C.*, as the case may be, for limits of the curve of adjustment.

3. From tangent, or (if compound or reversed curve) from either shift curve produced, lay off deflections, and at distances as shown in table, taking, however (if both curves have been shifted), for compound

* *P. C.*, point of commencement, or end of curve; *P. C. C.*, point of change of, or compound curve; *P. R. C.*, point of reverse curve.

curve the column showing difference of degree of two curves, and for reversed curve the column showing the sum of degree of the two curves. If preferred, half the adjustment curve may be laid off from one end and half from the other, using only the first half of the column.

4. In case of other curve than shown in the table, multiply the column for 1 degree curve by the degree (and fraction) of the desired curve.

5. Centers can be put in 10, 20, 40 or 50 feet apart as deemed sufficient, or at intermediate distances on trestles, bridges, etc., wherever necessary, by interpolation in the table.

6. A table of ordinates 5 feet apart is added for more correctly and conveniently setting out uniform curves from their hundred-foot chords.

7. Proper elevation for speed of 38 miles per hour is the middle ordinate of 60 feet of rail on any curve = .078 of a foot per degree, or $\frac{9.4}{100}$ of an inch.

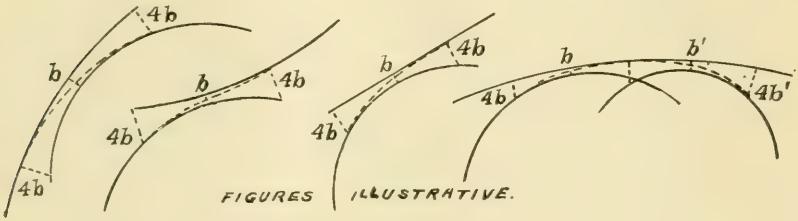


TABLE No. 1.—Distances and Deflections, Etc., for Flattening Curves in feet and decimals.

Distances.	Curve 1 degree.	2 degrees.	3 degrees.	4 degrees.	5 degrees.	6 degrees.	7 degrees.	8 degrees.	9 degrees.	10 degrees.	11 degrees.	12 degrees.
10	.0001545	.000	.000	.001	.001	.001	.001	.001	.001	.002	.002	.002
20	.0011634	.002	.003	.005	.006	.007	.008	.009	.010	.012	.013	.014
30	.00393	.008	012	.016	.020	.024	.028	.031	.035	.039	.043	.047
40	.00931	.019	.028	037	.047	.056	.065	.074	.084	.098	102	.112
50	.01818	.036	.055	.073	.090	.110	.127	.145	.164	.182	.200	.218
60	.03141	.063	.094	.126	.157	.188	.220	.250	.283	.314	.346	.377
70	.04989	.100	.150	.200	.250	.300	.350	.400	.450	.500	.550	.600
80	07446	.150	.223	.298	.372	.447	.520	.596	.670	.745	.820	.894
90	.10603	.212	.318	.424	.530	.636	.742	.848	.954	1.060	1.166	1.272
100	.14544	.290	.436	.582	.727	.873	1.018	1.164	1.310	1.454	1.600	1.745
110	.19358	.387	.580	.774	.968	1.160	1 355	1.550	1.742	1.936	2.130	2.323
120	.25132	.503	.754	1.005	1.257	1.508	1.760	2.010	2.262	2.513	2.765	3 016
130	.31953	.640	.960	1.278	1.598	1.917	2 237	2.556	2.876	3.195	3.515	3.834
140	.39909	.798	1.197	1.596	1.995	2.395	2.794	3.193	3.592	3.990	4.390	4.790
150	.49086	.982	1.473	1.963	2.454	2.945	3.436	3.927	4.418	4 910	5.400	5.890
160	.59573	1.190	1.787	2.383	2.980	3.574	4.170	4.766	5.362	5.957	6.553	7.150
170	.71455	1.430	2.144	2.858	3.573	4.287	5.002	5.716	6.430	7.146	7.860	8.575
180	.84821	1.696	2.545	3.393	4.240	5.090	5.937	6.786	7.634	8.482	9.330	10.18
190	.99758	1.995	2.993	3.990	4.988	5.985	6.983	7.980	8.978	9.976	10.97	11.97
200	1.1635	2.327	3.490	4.654	5.818	6.980	8.145	9.308	10.47	11.64	12 80	13 96
Shift curve....	0.2903	0.58	0.87	1.16	1.45	1.75	2.04	2.33	2.62	2.91	3.20	3.49
Shorten chain..	0.005	0.020	0.046	0.08	0.13	0.18	0.25	0.32	0.41	0.51	0.61	0.73

TABLE No. 2.—ORDINATES OF UNIFORM CURVES—IN FEET AND DECIMALS.

DISTANCES.	Curve 1 degree.	2 degrees.	3 degrees.	4 degrees.	5 degrees.	6 degrees.
5	.0414	.083	.124	.166	.207	.249
10	.0785	.157	.236	.314	.393	.470
15	.1112	.222	.334	.445	.556	.667
20	.1396	.280	.420	.558	.698	.838
25	.1636	.327	.490	.655	.818	.980
30	.1832	.366	.550	.733	.916	1.100
35	.1985	.397	.596	.794	.943	1.190
40	.2094	.420	.628	.838	1.047	1.256
45	.2160	.432	.648	.864	1.080	1.296
50	.2182	.436	.655	.873	1.090	1.310

DISTANCES.	7 degrees.	8 degrees.	9 degrees.	10 degrees.	11 degrees.	12 degrees.
5	.290	.332	.373	.415	.456	.497
10	.550	.628	.707	.785	.864	.942
15	.780	.890	1.000	1.112	1.224	1.335
20	.977	1.117	1.256	1.396	1.536	1.675
25	1.145	1.310	1.473	1.636	1.800	1.964
30	1.293	1.466	1.650	1.832	2.016	2.200
35	1.390	1.588	1.787	1.985	2.184	2.382
40	1.466	1.675	1.885	2.094	2.303	2.513
45	1.512	1.728	1.944	2.160	2.376	2.592
50	1.527	1.746	1.964	2.182	2.400	2.618

FORMULAS AND EXCEPTIONAL CASES.

For speed of 40 miles per hour, length of adjustment curve is equal to 25 to 30 times degree of curvature (sum of degrees if reversed curve, difference if compound)—or in letters, $a = 25\ c$ (or $c + c^1$, or $c - c^1$) . . (1)

Take it, however, as a convenient even number, then the shift for each curve will be 8 times the square of length of adjustment curve, multiplied by degree of curvature, divided by

$$1\ 100\ 000;\ \text{or}\ s = \frac{8\ a^2\ C}{1\ 100\ 000} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The deflection distance (d) of adjustment curve at any point distant x from the end of adjustment curve will be

$$(d) = \frac{4\ x^3}{a^3}\ s\ (\text{or}\ s \pm s^1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

To connect with an adjustment curve, detached curves, or curve and straight line detached, first shift the curve or curves, according to the table, or otherwise, if occasion require, by formulas, so as to allow of flattening other ends. Mark point where two lines are parallel—found half way between points where the two lines are equi-distant from each other. Measure the distance (b) between lines at the parallel point.

The ends of adjustment curve will come where the two lines are $4\,b$ apart, and in this case the length of it (a) must be taken as it comes, and not evened as above; or its length can be calculated if points distant (l) from each other have already been found where lines are equi-distant (e)—then $a = l\sqrt{\frac{3\,b}{e-b}}$ .....(1 a)

The deflection (d) at any distance (x) is found from $d = \frac{4\,b\,x^3}{a^3}$. .(3 a)

This last-above method will apply when curves in reverse direction have less than 200 feet of straight line connecting.

In case of curves in same direction—intersecting if produced, and not admitting of connection, by 200 feet or more of tangent—connect them by 200 feet or more of lighter curve than either, then shift all three curves and proceed by table.*

DISCUSSION.

W. Howard White, M. Am. Soc. C. E.—A very simple and satisfactory way of easing the ends of curves, which may be novel to many Members, consists in shifting the whole curve a uniform distance towards its center—that is, reducing its radius—after the road-bed is ready for the track. This has the effect of prolonging the curve at each end, and at the same time of easing the curvature in the end stations of the original curve.

Thus for a 5-degree curve, if we set the whole curve over ½ foot to the inside, and reduce this 25 per cent. at the original tangent points, the latter will have 0.38 foot deflection from the tangents. Now, if the tangent points are assumed as 100 feet from the old ones back on the tangents, the curve will begin with a 26-minute curve for one station. The 26 minutes deflected in this station is, of course, taken out of the 5 degrees, which would otherwise have been turned in the next 100 feet, making the angle in the same 4 degrees 34 minutes. The third station has the regular deflection of 5 degrees (all angles referred to are center angles).

The advantages of this method are three-fold. In the first place, it

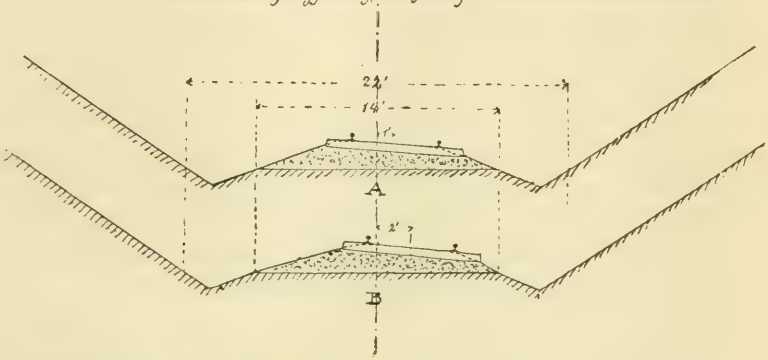
*The above formulas, etc., are partly adapted from Rankine.

avoids the extra work of putting in a compound curve instead of a simple one on given tangents. Secondly, it avoids compounding the curvature of the located line, and makes it much easier to mark the line permanently by monuments and to recover it, if a dispute arises as to right-of-way boundaries. Thirdly, the track lies better in the cuts and fills for maintenance when thus decentered.

I might add that it is also a method perfectly applicable to easing curves on completed lines where this point has not been looked out for in the construction.

The correctness of the third point of advantage gained will be seen on reference to the following figure. In *A* the track is represented shifted as here recommended, and elevated 0.04 feet per degree for a 10-degree curve.

Illustrating Effect of Shifting Curve to Flatten its Ends



The center of tracks is moved 1 foot towards the curve center.

The outer ends of the ties, with the assumed elevation, will be 4 inches approximately higher than the center, or (with 1 foot of ballast under the ties at the center) will be 16 inches above the road-bed. Distance to edge of ditch, as shown, 4 feet. Slope, 3 to 1. The inner end of the tie is 8 inches above the road-bed. Distance from ditch, 2 feet. Slope, 3 to 1.

So, while easing curve, we equalize ballast slopes in cuts, and it will be readily seen that the same applies on fills equally.

Of course with stone ballast this does not apply equally, but still it is of some advantage to have more room outside of the tie even with stone ballast, and the other advantages remain the same.

As to the amount of the above shifting, it will be perceived that the

basis for it in the examples given is 1½ times the difference of elevation of the two ends of the tie, or about 2½ times the super-elevation of the outer rail.

This proportion would be determined exactly by the width of road-bed, depth of ballast, and super-elevation used, if one wished to maintain a symmetrical section of the form I have shown at *A*.

I prefer, however, to use about double this—giving as follows:

	1° Curve.	5° Curve.	10° Curve.
Shift of tracks.....	0.20 feet	1.00 feet	2.00 feet
Offset of tangent point...	0.15 "	0.75 "	1.50 "
Angle in first station	0°10′	0°51′	1°42′
" " second "	0°50′	4° 9′	8°18′
" " third "	1°	5°	10°

I have shown at *B* the effect of this shifting for a 10-degree curve with the same elevation as before. It reduces the inner ballast slope to 1½ to 1 with the given road-bed, and if the outer be carried up 3 inches on the tie it will be a little flatter than 3 to 1.

The increased amount of shifting is desirable in order to give the entrance curve a more uniform increase of curvature, and its effect on the ballast appears to me not undesirable, since the outer slope should be flatter on account of its greater liability to wash—from carrying more drainage—and because it is on that side that stability against side thrust and vertical pressure of the train is needed.

The curvature in the second station of my tabulated examples is not uniform. The angle given is the total turned in the station. In reality the second station of the 10-degree example is made up of 35 feet of approximately 5-degree curve, and 65 feet of 10 degree.

In curves above 7 degrees it is desirable to carry the easing still further, by slightly offsetting the first station on the tangent. Curves eased in this way ride remarkably well.

The idea of easing a curve by offsetting had occurred to me before I knew of its being practiced elsewhere, and I first applied it by offsetting the whole construction, having the curved stakes moved over before the slopes were staked out.

The advantage of doing it on the completed road-bed was first pointed out to me by a letter on the subject to the *Railroad Gazette*, some years ago, from a gentleman whose name I unfortunately do not remember.

This method is particularly applicable to the relief of reversed curves.

Any superintendent who has one of these railroad racks on his division, who will take the trouble to have the whole of each of the curves moved inward and the track well lined between, will probably be astonished at the improvement.

A. M. WELLINGTON, M. Am. Soc. C. E.—In the *Railroad Gazette* of March 11th, 1881, I published a method for putting in transition or connecting curves by offsets, which gives in substance the same curve as that proposed by Mr. Man and Mr. White, viz.: a cubic parabola, but in which the method used was somewhat different, and, I think, preferable, requiring no shifting of the circular curve after it was once run, but running it in, in the beginning, at an offset from the tangent, after doing which a few stakes only, near the *P. C.*, need to be set over to complete the transition curve.

This method, however, like that of Mr. Man's and all others with which I am familiar, was incomplete in this : that it required a certain fixed offset for each curve. This is sometimes difficult to secure unless the offset is (as I think is the case with Mr. Man's method) made objectionably small ; in addition to which, by expanding the method, so that transition curves of any length and offset may be used with any curve, we gain the great advantage that nice adjustment of the line to the topography of the country is greatly facilitated, and much time save din connecting new and old lines together when revisions have been introduced.

I therefore devised, while in Mexico, a method, which was extensively used on the lines under my charge in that country, for using any transition curve which the topography or other cause made most convenient with any curve. I consider this the preferable system by far for using such curves, since, so far from adding to the difficulties of the field-work of location, it so materially reduces them that it is easier to use transition curves than not to use them, while it enables us to use, wherever circumstances permit, long and easy transition curves, without impeding the use of shorter and quicker curves when necessary. I regret that it is out of my power to include a more formal description of this method in this discussion, but I hope to lay it before the Society at no distant day, contenting myself in the meantime with objecting to any method which requires always a fixed offset as inadequate to the requirements and unnecessarily limited.

Neither do I consider that the real reasons for invariably using such curves are always correctly put. An incidental advantage of them is that it enables the elevation of the outer rail to be always in correct ratio (or, at least, in uniform ratio) to the radius of the curve at each point, while passing from the tangent to the curve, and *vice versa;* but this gain, while undoubted, is only incidental and of minor importance. The chief purpose of such curves is (1) to avoid the shock incident to the almost instantaneous turning of the truck upon the center-pin, and especially the sudden jerk to the wheel-base of the locomotive; and (2) to avoid the sudden generation of great centrifugal force, with its attendant lateral jerk, and permit this force to be generated and applied gradually. For both of these latter purposes the curve can never be too long; and, therefore, should not be unnecessarily shortened in one place by any procrustean rule, because it may be essential to do so in another place. As a general rule economy of construction is materially promoted by the use of easy transition curves, for the reason that they are apt to more correctly fit the natural contour in swinging around a curve or a hollow.

My attention was drawn to this subject many years ago, by observing how absolutely invariable a rule it is, on old track in good condition, to find that the trackmen have extended the curve, as the engineer left it, back on to the tangents, usually for two or three hundred feet, by throwing the line inward at the tangent point, thus necessarily sharpening the curve somewhat at some point beyond the tangent point, but accomplishing the far more important end of easing the approach. This has been found essential by all trackmen to obtain easy riding track, and on hundreds of old curves whose existing alignment I have determined I have never found it absent. The same phenomenon has been observed by many other engineers on other roads, and accordingly it is among the engineers of lines in operation that we find the greatest efforts made to introduce such curves, while engineers who have been engaged chiefly or wholly on construction are apt to regard them as a rather pedantic refinement. I shall endeavor to avail myself of an early opportunity to lay the method I have referred to before the Society.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

332.
(Vol. XV.—May, 1886.)

ON CRANES AS LABOR-SAVING MACHINES.

By C. J. APPLEBY, M. Inst. C. E.
READ OCTOBER 17TH, 1883.

WITH DISCUSSION.

As a well-constructed crane, or other similar power machine requiring one man to drive it, will not only do as much work as can be done by ten men performing similar work by manual power, but in one-tenth of the time they would require, it seems singular that railroad and water-side depots should so rarely be laid out with a view to the immediate or ultimate employment of labor-saving machines of this kind. The result of this is, that when the pressure of increasing traffic comes, as it nearly always does, expedients have to be resorted to and expense incurred, both of which might have been avoided if proper provision had been made in the first instance.

These remarks apply with equal force to workshops, and to many works of construction, where the time and cost of handling materials is always a most important element.

Using the word "crane" as a short and convenient term for almost any kind of lifting machinery which may be generally used under the conditions above referred to, the considerations which seem to present themselves are:

First.—As to the best mode of applying or transmitting power; and

Second.—Whether machinery of this kind should be fixed, or, as far as possible, portable.

It may be well here to call attention to the self-evident fact, that the most economical working result is obtained from machines which are so arranged that when they take hold of the load it is not released until it has been deposited where it is required—such as from ship to car, or the reverse; from car to van for distribution; or, in workshops, from the machine to the erecting floor, and so on.

The question of transmission of power has been referred to in some papers read before the Institution of Civil Engineers (England), and in the Transactions of some of the technical societies on the Continent of Europe, but, so far as the writer knows, the subject has never been exhaustively treated, although those who have been engaged in crane construction possess a considerable amount of information as to the conditions under which the different systems can be most economically employed.

The systems of transmitting or applying power are:

1. The well known hydraulic system, with pressure pumps, accumulator and distributing pipes.

2. Compressed air distributed through pipes.

3. Steam, distributed as above.

4. High-speed rope, or "endless cotton cord," which runs at a speed of 5 000 to 6 000 feet per minute.

5. Low-speed rope, running 1 500 to 2 000 feet per minute.

6. Square shaft supported on tumbler bearings.

7. Steam from a boiler delivered on the top of a piston, with multiplying chains similar to the hydraulic system.

8. Boiler and engine fixed on the crane, and driving gear for the several motions required.

A consideration of these systems will lead to the conclusion that those numbered 1, 2 and 3 can only be applied to fixed cranes, or to such as have to be moved over a very limited area.

It will be evident that the arrangements enumerated under Nos. 4, 5 and 6 will transmit power over a very large area, but it should be rectangular, or nearly so; hence the use of this system has been limited to working overhead traveling cranes, driving capstans, and a few other and similar purposes.

The only system which can be used universally, wherever there is a railway track, is that referred to under No. 8.

The hydraulic system possesses indisputable advantages over compressed air or steam, and, although there are some drawbacks, it has been so extensively and successfully used, that the objections againstit go for very little when considerations of climate need not be taken into account. Experience in countries where the cold is far less severe than it is in the busy parts of this continent, points to the conclusion that the common use of hydraulic power will be attended with considerable inconvenience when winter begins to set in.

Compressed air has been used under very widely differing conditions, but it is considered by many to be a somewhat expensive mode of transmitting power. There seems no reason why it should be much more costly than hydraulic power, but, as a matter of fact, its use has been abandoned in many cases.

Steam is rather largely used for working warehouse hoists, capstans, cranes, etc., and it is common to carry it through 1 000 feet of pipe without further inconvenience than that incidental to the heat radiated from the pipes, and to the slight condensation which is inevitable, however well the pipes may be protected.

The high-speed cotton cord, as introduced and so successfully used by Mr. Ramsbottom, runs at a speed of 5 000 to 6 000 feet per minute. As is well known, the cord works in V-grooved pulleys, very accurately turned and balanced, the power being transmitted from one end of a building or shop. The cord is carried on rollers, or other supports, at intervals of 10 to 25 feet, passes over grooved pulleys on the machine to be driven, and finally over another grooved pulley, which is weighted so as to keep the cord in tension, and thus give the power required for working. The cost of the cotton cord is often considered a serious item; but probably the wear and tear complained of is due far more to negligence in fixing the supports, or to maintaining these high-speed bearings, than to any defect inherent in the design. In some cases the different motions are taken from the cord, which, in that case, takes the place of the shaft, as shown in Plate No. XV. But the writer believes the latter is better construction, and but little more costly. It will be understood that all cranes of this type are worked by one man, whether they be 5 or 50 tons power.

Low-speed rope transmission differs from that last described only in

the speed of the rope, which is 1 500 to 2 000 feet per minute. Any good hemp rope, usually 1 inch to 1½ inches diameter, answers every purpose, and if fairly well looked after will last a long time; but it is desirable to change the ropes at intervals of one to two years. The speed is sufficient to give the necessary driving power, but evidently the tightening gear must be heavier and stronger than that required for the higher speed rope or cord.

The square shaft (Plates Nos. XVI and XVII) has been used for many years, but to a less extent than it might have been, owing to the difficulty experienced in properly supporting the long line of main driving shaft from which all the motions are transmitted. There have been many devices for overcoming this difficulty, but probably none of them fulfill the requisite conditions so completely as that shown in Plate No. XVII. There is a striker plate on the end cradle of the traveler which causes the strut to fall into the position shown. The curved bearing bracket is hung on a pin and follows the straight arm until the driving pulley or geared wheel (as the case may be) has passed over the bracket. The other end of the striker plate then restores the bracket into its original position so that only one bearing can be down at any one time; the main shaft being, in the meantime, supported by the sleeve through which it has to pass.

The question as to the relative advantage of rope and shaft transmission is naturally very largely influenced by local circumstances. As a general rule, but one open to many exceptions, the rope system costs less and is better than the square shaft where the distance for transmitting power exceeds about 200 feet. Below that distance the shaft is probably the best, and certainly the cheapest. But machinery has often to be driven at different levels or at an angle with the point from which the motive power is transmitted. In such cases the rope possesses manifest advantages, because it can be carried by guiding pulleys in any direction required, one set of driving gear and one tightening apparatus sufficing for a great length of rope.

The steam crane referred to under system No. 7, is that known as the "steam hydraulic" system, and is worthy of mention as one of the modes in which power has been applied to portable cranes. The construction generally resembles that of the well known hydraulic crane, with multiplying chains, etc., steam taken from a boiler on the revolving platform being used instead of hydraulic pressure. It has found but little favor

with users of such machinery, and, so far as this country is concerned, it seems open to much the same objection as has been suggested with reference to the use of hydraulic power in cold weather.

The type of crane most universally employed under widely differing conditions, is that illustrated in Plates Nos. XVIII, XIX and XX, probably because machines of this type perform more functions than can be obtained from any other mechanical arrangement applied to lifting and placing loads in any desired position. All cranes of this kind should lift and turn round by steam power. That in Plate No. XIX has additional motions for altering the radius of the jib, and for traveling along the track, also capstans attached to the under carriage. These capstans are used for hauling materials so as bring them within the radius described by the jib of the crane, or for drawing up empty and shunting loaded cars, thus dispensing with the use of locomotive, horse or manual power, which, in the absence of this appliance, would be required. Machines of this type are made from 2 to 10 tons power, and they possess the advantage of each being self-contained and complete in itself, so that the number may be indefinitely increased, as and when required; they can also be distributed or concentrated to meet the exigencies of traffic for the time being.

As regards fixed cranes for loading and discharging ships, we frequently see one-third, or even one-half, of the total number of cranes, erected at a large cost at intervals along a dock or basin, entirely idle, and a considerable length of quay unoccupied, simply because certain ships have had to be berthed in such a way that the largest number of cranes can be concentrated on them. This is obviously due to the fact that when the positions of the cranes were decided, it was impossible to foresee what length of ships would be berthed opposite to them, the distance between hatchways, and so on. It would, therefore, seem that—for a given outlay—the greatest duty will be obtained from cranes which can be moved to any place where their services are required.

A very successful arrangement is where two or more railroad tracks run parallel with the water frontage, and where the cranes are of the type shown in Plate No. XXI; these discharge from the ship into the cars or to a depot beyond.

It will generally be convenient to make the gantry to span two lines of track, allowing clearance for the highest vehicle usually passing under it, the rails on which the structure travels being outside the car tracks.

By this means the quay space immediately adjoining the water, always the most valuable, and usually the most costly, is practically not trespassed upon. Cranes of this type are generally of 3, 5 or 7 tons power; for, although the bulk of the packages to be handled will be less than one ton, considerable strength is required in the revolving bed, etc., to withstand the strains incidental to a radius of 30 feet, or even more, which is often necessary for delivering the load from one point to the other without rehandling. It is obvious that if a few connections are put in between the two tracks, either may be used for loaded or empty trucks, and so facilitate removal and marshaling. It will be observed that the radius of the jib can be adjusted to reach the center of the hatchway; this, with the facility of moving along the track (both operations being performed by the driver), enables him to place the crane in such a position that the jib will clear the rigging when revolving. The capstans at the foot of the main framing are driven by the engine, and are used for hauling trucks as already described. The convenience of the driver being at a sufficient height to have a clear view of his work, notwithstanding the rise and fall of tide and variations in the height of the ship above water line as she becomes lighter, recommends this system for use under many conditions. Several modifications have been used, such as having an overhead road on which the crane travels. In some cases this has been made of wide span and the crane has traveled transversely as well as longitudinally. But the circumstances under which these motions are requisite do not often arise.

Plate No. XXII shows a mode of discharging and loading ships which may often save a considerable outlay—as it did in the case for which that design was made—for dredging and for better foundations than are required for merely carrying loaded cars. The crane is fixed at one end of a craft of sufficient beam to give the required stability when the jib is at a right angle with the length of the craft, the other end being used as a store and for sorting for different destinations.

Floating cranes of large power, such as are illustrated by Plates Nos. XXIII and XXIV, are less used than they might be, although it seems probable that a larger duty will be obtained from them than from fixed cranes of similar power. Where there is ample space for manœuvring, the arrangement shown in Plate No. XXII. answers every purpose; but for use in docks, or where space is limited, the design in Plate No. XXIII is certainly more convenient. They are made up to 60 tons power, and this might be exceeded if required.

Locomotive cranes, Plate No. XX, are made of varying powers, from 10 to 25 tons, and it will scarcely be necessary to point out how usefully they may be employed about foundries, railway repairing shops and similar works, and they are also available for use in case of accident on any part of the line. It will be seen that the motions are similar to those of the crane shown in Plate No. XIX, and it may be well to mention that light work can be done at high speeds, these being arranged to coincide, as far as possible, with the time requisite for handling. The under-carriage is always made to suit the rest of the company's rolling stock, that shown in the diagram being for a railway on the Continent of Europe.

As regards terminal freight stations, the arrangements differ so widely that no general rule can be established; but, so far as the writer's observation goes, the greatest economy in time and cost has been achieved in depots where the platforms are rather narrow, and have fixed cranes so spaced that one will reach to the center of two cars. There are similar cranes on the other side of the platform, by preference so placed that the jibs will intersect the circles described by those of the cranes on the opposite side. By this arrangement there is a better chance of separating merchandise for different destinations, and the labor incidental to separating it is proportionately reduced. Whether the cranes should be worked by hand or by power depends almost entirely on local conditions, but as the bulk of such merchandise consists of comparatively light packages, probably in most cases an inexpensive hand-power crane, such as that indicated by Plate No. XXV, will answer every purpose. These cranes command the center of the open cars, and reach to the center of the doors of box cars. The labor of "breaking down" in the latter and landing the merchandise on the platform or on a plank between the car and the platform is found to involve far less labor, and is done by cranes in less time than is occupied by trucking. But these remarks apply exclusively to depots which have been designed with a view to the use of mechanical appliances, and where the general arrangements have been made to suit them.

The duty performed by these appliances varies according to the nature of the material, but the working speed of a well-constructed crane being far in excess of the speed at which the loads can be brought within range of the jib, it is this that regulates the output, not the working speed of the crane.

As much as 80 tons per hour have been lifted a height of 20 to 30 feet, and deposited at a distance of 60 feet from where it was taken up, the loads being 1½ to 2 tons each. But men could not handle continuously at that speed, and 40 tons per hour is considered a large duty. The crane was of the type shown in Plate No. XVIII. A similar crane commonly delivers 240 barrels of oil per hour under similar conditions as to height of lift, etc. The cost of working is one driver's wages, a few hundred pounds of coal per day for each crane, some oil, wipings, etc. Five per cent. per annum is an ample allowance for depreciation.

Many illustrations of the advantage of being able to concentrate crane power might be given, but two will suffice. Three cranes of the type shown in Plate No. XXI discharged a steamer carrying 2 000 tons of hematite ore, and loaded her with 1 200 tons of wrought-iron pier-work in less than thirty-six hours. In another case two cranes, similar to that shown in Plate No. XVIII, loaded up 2 000 tons of cast-iron pipe on railway cars in less than forty hours.

The cost of this system of working, as compared with manual labor, is easily ascertained; but it would be difficult to estimate at its proper money value the larger gain arising from the increased carrying capacity obtained from a given quantity of rolling stock, or from a given tonnage of ships, and from passing a larger quantity of merchandise through the depot or dock. Evidently the same conditions apply to workshops, or any place where large quantities of materials have to be handled.

The machines referred to in this paper have been designed under the writer's immediate supervision, and have been made by his firm.

DISCUSSION.

C. E. Emery, M. Am. Soc. C. E.—This paper is a revelation in a department which has not been developed in this country as yet. The more general introduction of labor-saving machines of this character will enable us to transmit freight at a saving. One reason the crane is used in railroad work in England, is that their cars are covered with tarpaulins, which gives a very great advantage, as the cranes can be swung over the cars. On account of our system of putting everything inside box cars, we have to pay extra for the transmission of freight into and out of the cars.

ROBERT CARTWRIGHT, M. Am. Soc. C. E.—I would ask Mr. Appleby whether, as a labor-saving material, hemp is better than cotton?

C. J. APPLEBY, M. Inst. C. E.—As to its being a better material, I hardly know in what sense you mean. Do you mean cheaper, or its capacity to do better work? The cotton rope does very well, but a more satisfactory thing is a good hemp rope. If you use a steel rope, the cost would be greater than that of hemp for a given period.

ROBERT CARTWRIGHT, M. Am. Soc. C. E.—Then you think the hemp rope is better than the cotton?

Mr. APPLEBY.—I have not used cotton except of $\frac{5}{8}$-inch diameter. I have never used cotton rope more than $\frac{5}{8}$-inch, and I have not found it more suitable than hemp. The cotton rope costs more with us. I don't know how it may be here.

Mr. CARTWRIGHT.—It is only the best cotton rope that would do. And speaking of that, the best is the cheapest. If a cotton rope costs so much per pound, but lasts longer than the steel rope, at the end of ten years the cotton is the cheapest.

Mr. APPLEBY.—To use a metallic rope you want to have a pulley corresponding with the diameter of the rope you are using. We get a fair-sized pulley; but where you have to deal with overhead traveling cranes you have to consider the question of head room. You can't afford to sacrifice much in the diameter of the pulleys. But the best result from a metallic rope of any kind has been with the drum.

Mr. CARTWRIGHT.—You can't drive that at very high speed.

JAMES PLATT.—Mr. Chairman, I am called on for my views by my friend, Mr. Appleby, and I must say at the outset that all my sympathies have been with the hydraulic system. Mr. Appleby's house has made a larger number and a greater variety of cranes than any other house in our little island. For many purposes I would prefer the square shaft where the length admits of it—say 300 feet. In case the rope is 5 000 or 6 000 feet, the cotton rope is best, but the diameter should be limited to $\frac{3}{4}$-inch. We have cotton ropes $\frac{5}{8}$-inch in diameter. They are doing very well. The cotton rope is undoubtedly better for high speed than hemp. Cotton is more simple. I believe that bearing pulleys of wood with iron bushes are best.

I believe, for places under cover, that the hydraulic system will be eventually tried in this country. There is still a very great scope for hydraulic cranes. I spent six months here last year and was very much

surprised to see so many workshops and depots without cranes, but no doubt you will come to it, and you have an illustration here to-night of the best plan adopted in our country. We employ steel wire rope for very many purposes. For driving cranes, etc., it does well, but unless very great care is taken of the pulley, you do a great deal of damage to the rope in a short time, and altogether it is not so good as cotton. As to hemp rope, we use ¾-inch at 600 feet.

Mr. CARTWRIGHT.—I would like to ask a question in relation to the square-shaft transmission. How long have you had it in actual use?

Mr. APPLEBY.—350 feet.

Mr. CARTWRIGHT.—When I speak of length, I mean the life of the shaft. How many years had you them in operation? This question will lead up to what I want to get at.

Mr. APPLEBY.—Well, I really don't know just at this moment, but I should think about twenty years. How long does a round shaft wear?

Mr. CARTWRIGHT.—We do not permit it to wear out; we replace it with another.

Mr. APPLEBY.—There is no difference between that and any other shaft, if properly proportioned. There is practically none. The great advantage of this particular system is that it doesn't matter about the length between bearings. You can have them 2 or 3 feet longer—any length you please—so that you can support your shaft to any extent you please. The life of the shaft depends upon the bearings. In this case I think you may put it down that the life of the shaft will be the same as any other transmission shaft which is properly proportioned. We have had a shaft running for forty years and it is running now.

I wish to say with reference to the question you have asked me about the life of the shaft, that I have not seen more than 350 feet of square shaft used, but that statement would probably mislead if I did not tell you that with that shaft, after very hard work, it was wonderful to see how it was twisted. Mr. Platt speaks of 300 or 400 feet as the length of the shaft. I do not agree with him, though his experience, and the intelligence he has brought to bear upon the question of hydraulic transmission, has contributed more than anything else to the success which the system merits; but I would rather put it at 200 feet, and I think the rope is cheaper and probably better for the long transmission—certainly cheaper.

THEODORE COOPER, M. Am. Soc. C. E.—The only crane I know of

worked by the square shaft is at the Dickson Manufacturing Company, of Scranton, Pa. I was there recently and made an examination of that crane. Its mechanism is very excellent. They have there a square shaft 200 feet long. They can carry 25 tons from one machine-shop to the other.

I would like to ask Mr. Platt in regard to the hydraulic system. Is it not an intricate method as compared with a direct use of power? Of course I acknowledge that for certain purposes the hydraulic system is good. But for general application is it not very complicated? Steam is applied to the pumps—water is pumped to the accumulator, thence conducted to the crane. Can this system of transmitting power be the best and most economical?

Mr. PLATT.—In speaking of the square shaft, perhaps I have in my mind, when I say 300 feet, where four shafts are in one piece. If a shaft is large enough to do the work it can be carried to that length, but at that length the rope is as good as the shaft. In regard to hydraulic transmission, the distinction is that you generate your power where you get a high duty of 90 per cent. In carrying that power, there is less loss in carrying high-pressure water. This is the distinction. In these cranes you have your rope running all the time, in the hydraulic system you only use the power in lifting and none in lowering. But in the power cranes you have all the time the power passing through the rope or shaft.

Mr. COOPER.—How much do you have to multiply? Don't you lose power?

Mr. PLATT.—No. If we multiply three times we have a certain proportion of cylinders. If we multiply four times we get less.

W. G. HAMILTON, M. Am. Soc. C. E.—What do you use in cold climates?

Mr. PLATT.—We put in glycerine.

R. L. HARRIS, M. Am. Soc. C. E.—I might mention that there are some beautiful hydraulic cranes at the Cleveland Steel Works.

Mr. COOPER.—I think you will have to keep the water warm. Glycerine will prevent freezing, but there is sure to be leakage. My experience has been this: that with portable riveters five hours out of ten were consumed in working, and the other five in packing the joints. It is impossible to keep the hydraulic riveter constantly in motion. It is a nice piece of mechanism, but, judging by my experience I would prefer some other riveter.

Mr. Appleby.—The hydraulic crane is a desirable thing where it can be used without any inconvenience, and in this country it can be used, particularly in steel works. Mr. Emery, in the course of his remarks, speaks about railroads using box cars, but it is very difficult to see how they can avoid it. Our system with open tops is all very well, and works well, but we have a comparatively short distance to run. Mr. Cooper mentioned a crane which he has seen. I am glad to hear his favorable opinion of it.

Mr. Hamilton.—You have not said anything about a walking crane, such as the Pennsylvania Railroad uses.

Mr. Appleby.—I have not mentioned them, because they are used very generally.

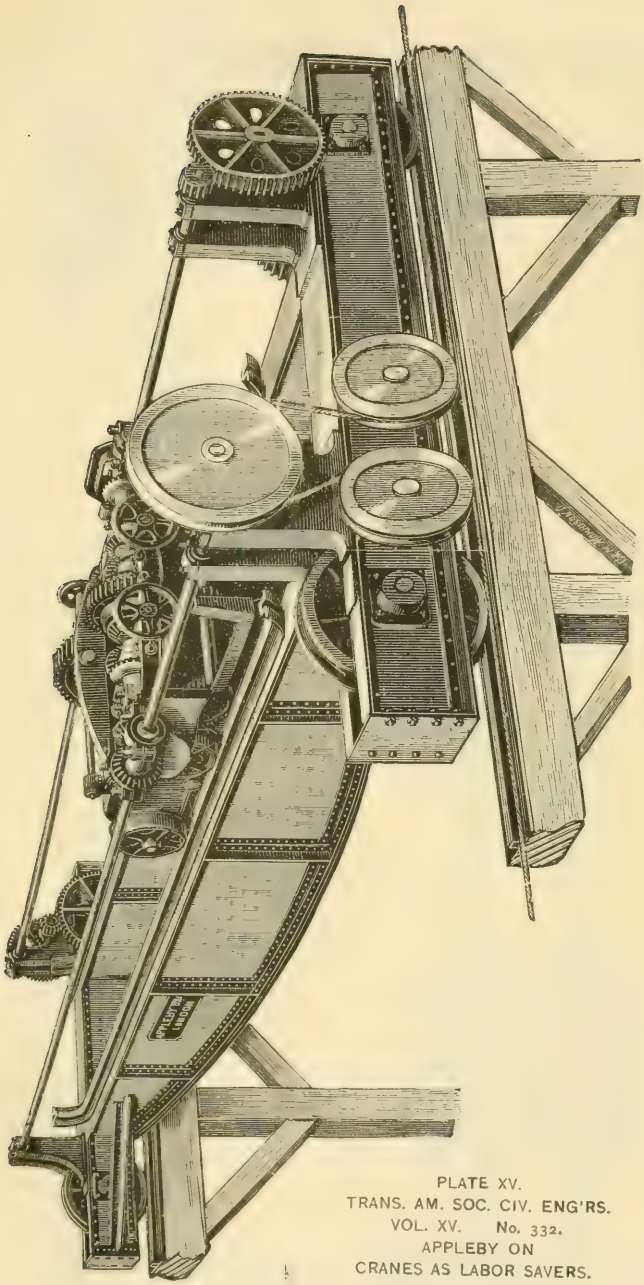
Mr. Hamilton.—Not in this country.

Mr. Appleby.—They perform the same functions as a traveling crane, but are placed only where it is utterly impossible to use a traveling crane. For instance, take a line of machines with room for a single crane to run. This crane runs up the line of railroad, drops the work alongside of the machine, and takes the other work away. You could not use any other appliance in that place. Those that I am building now are driven by power, but in a great many cases a hand-driven one answers the purpose.

In the floating crane we used water to counterbalance the crane. When the crane is run into position, while they are putting the load in slings, we set a pump to work pumping into tanks, and as soon as we find that the chain is getting taut we know we have our proper weight, and stop. It is desirable to use for most of this work very large floating cranes.

George S. Greene, Jr., M. Am. Soc. C. E.—In the 40-ton floating derrick, do you move the whole float?

Mr. Appleby.—Yes. In some wharves I think it is better to have a propeller, because it can be used so much easier. For long distances it would be used under any circumstances, and I believe it works remarkably well.



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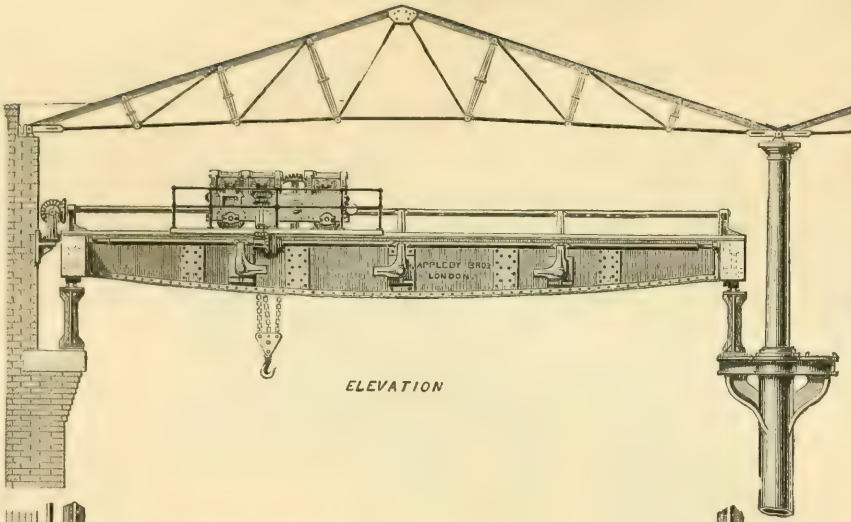
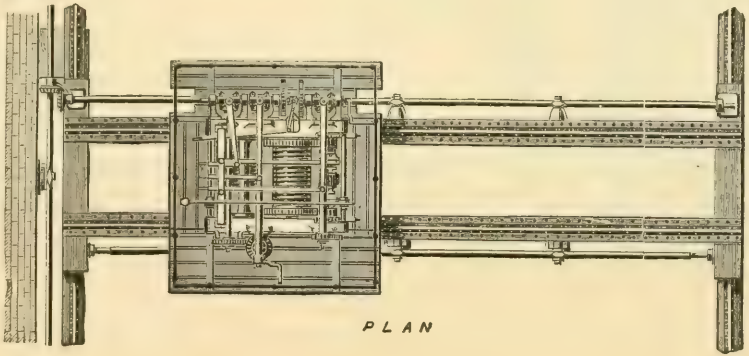
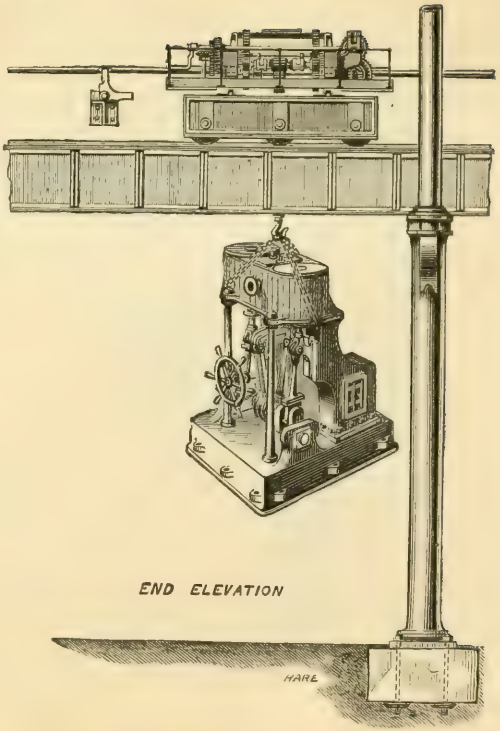


ELEVATION



PLAN



END ELEVATION

PLATE XVI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. No. 332.

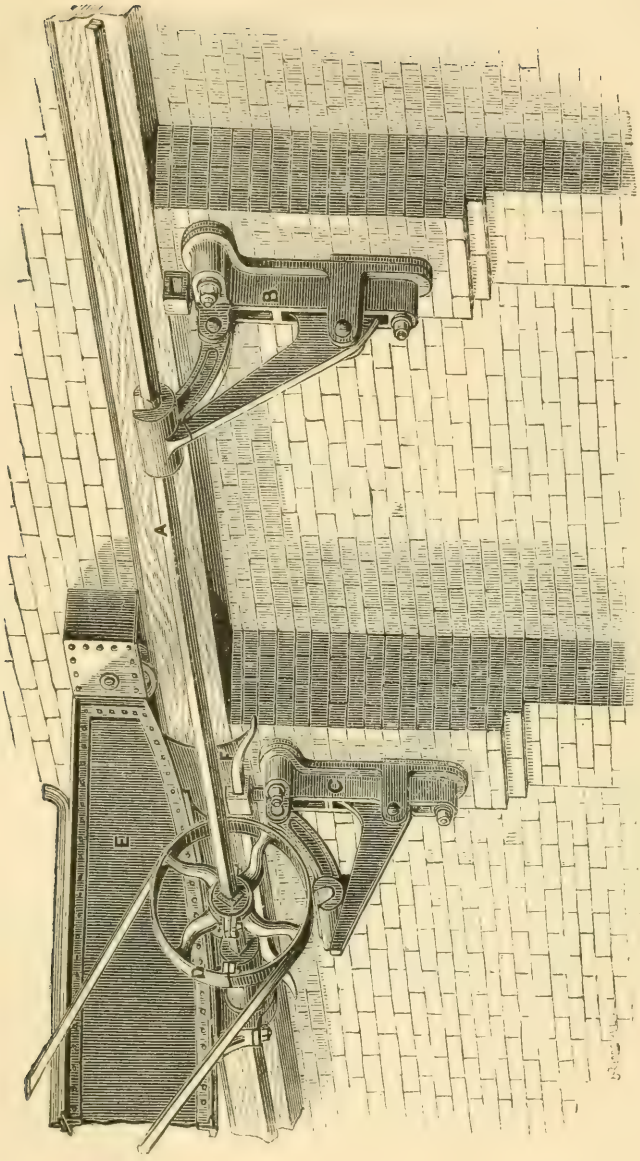


PLATE XVII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. No. 332.
APPLEBY ON
CRANES AS LABOR SAVERS.

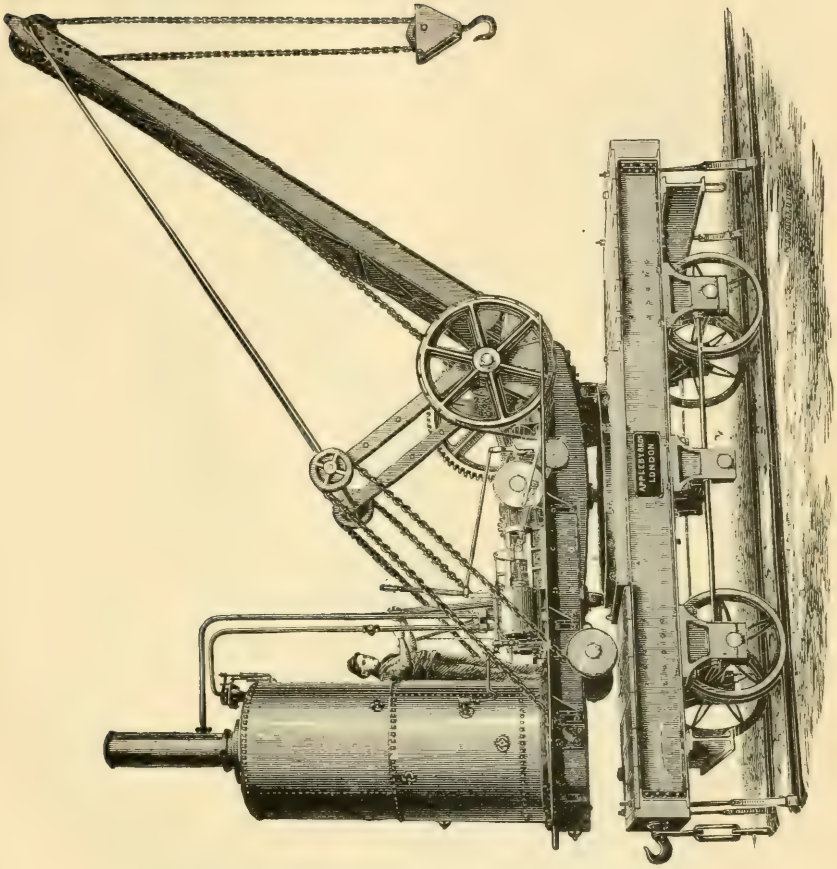


PLATE XVIII
TRANS. AM. SOC. CIV. ENG'RS.
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APPLEBY ON
CRANES AS LABOR SAVERS.

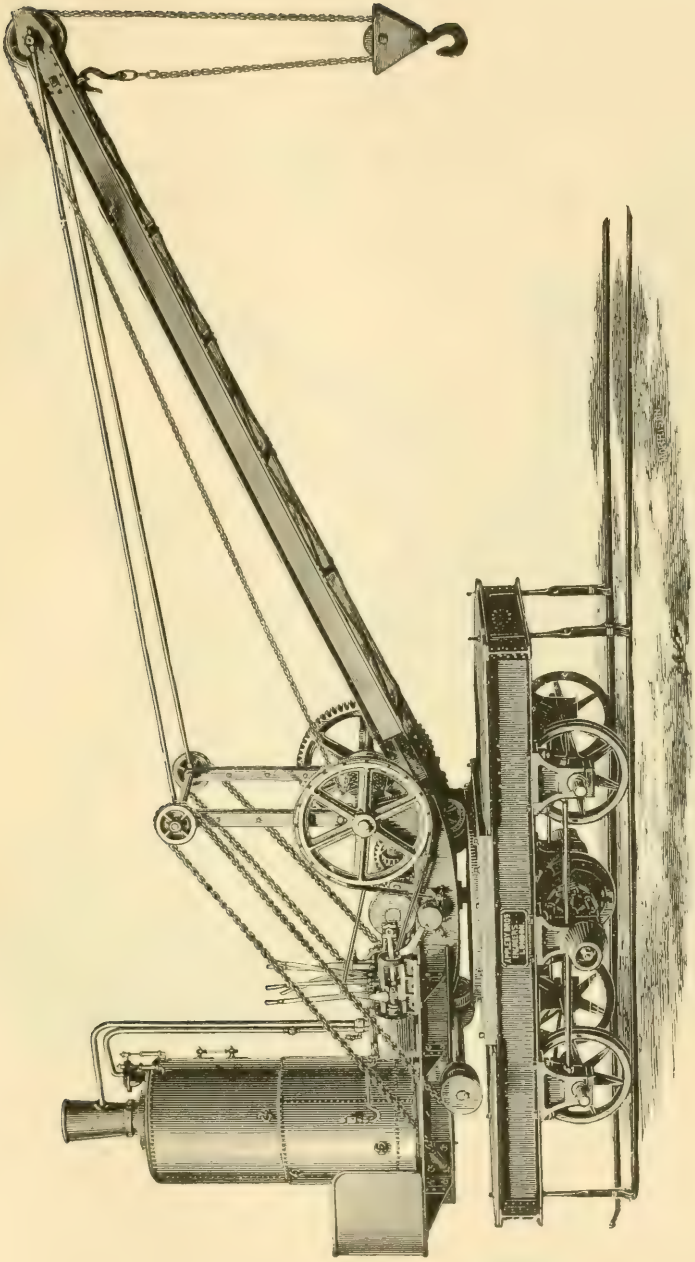

PLATE XIX.
TRANS. AM. SOC. CIV. ENG'RS.
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APPLEBY ON
CRANES AS LABOR SAVERS.

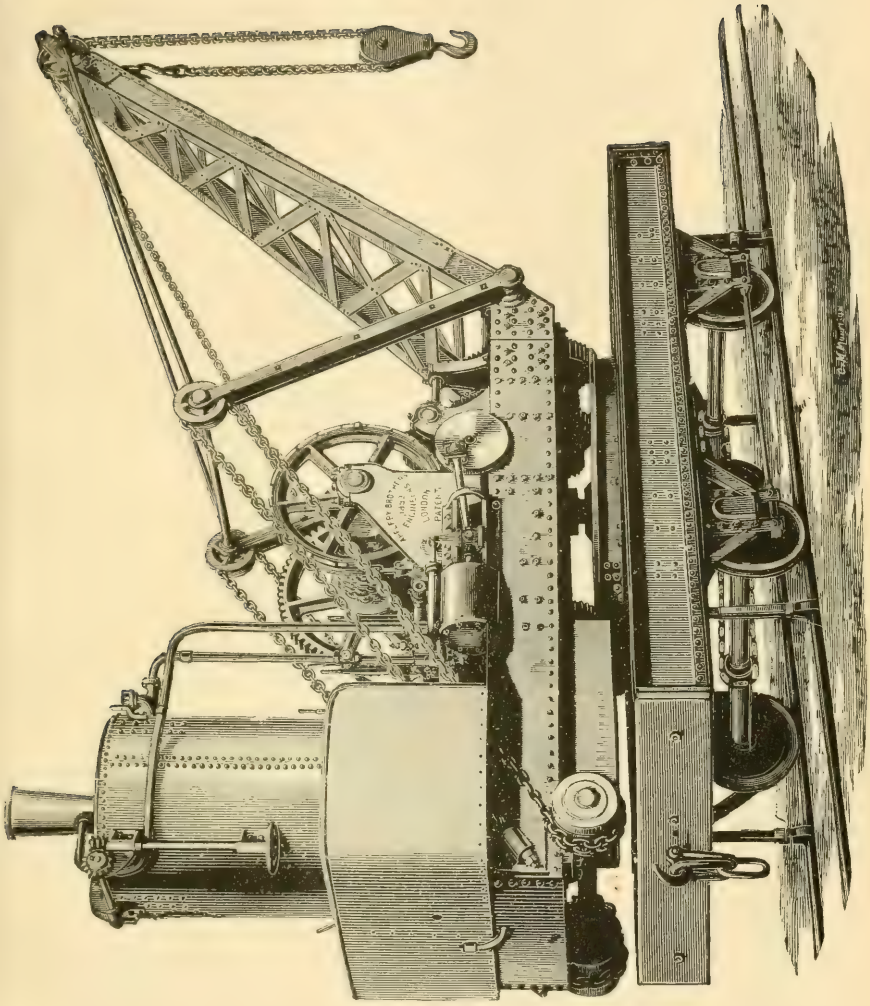
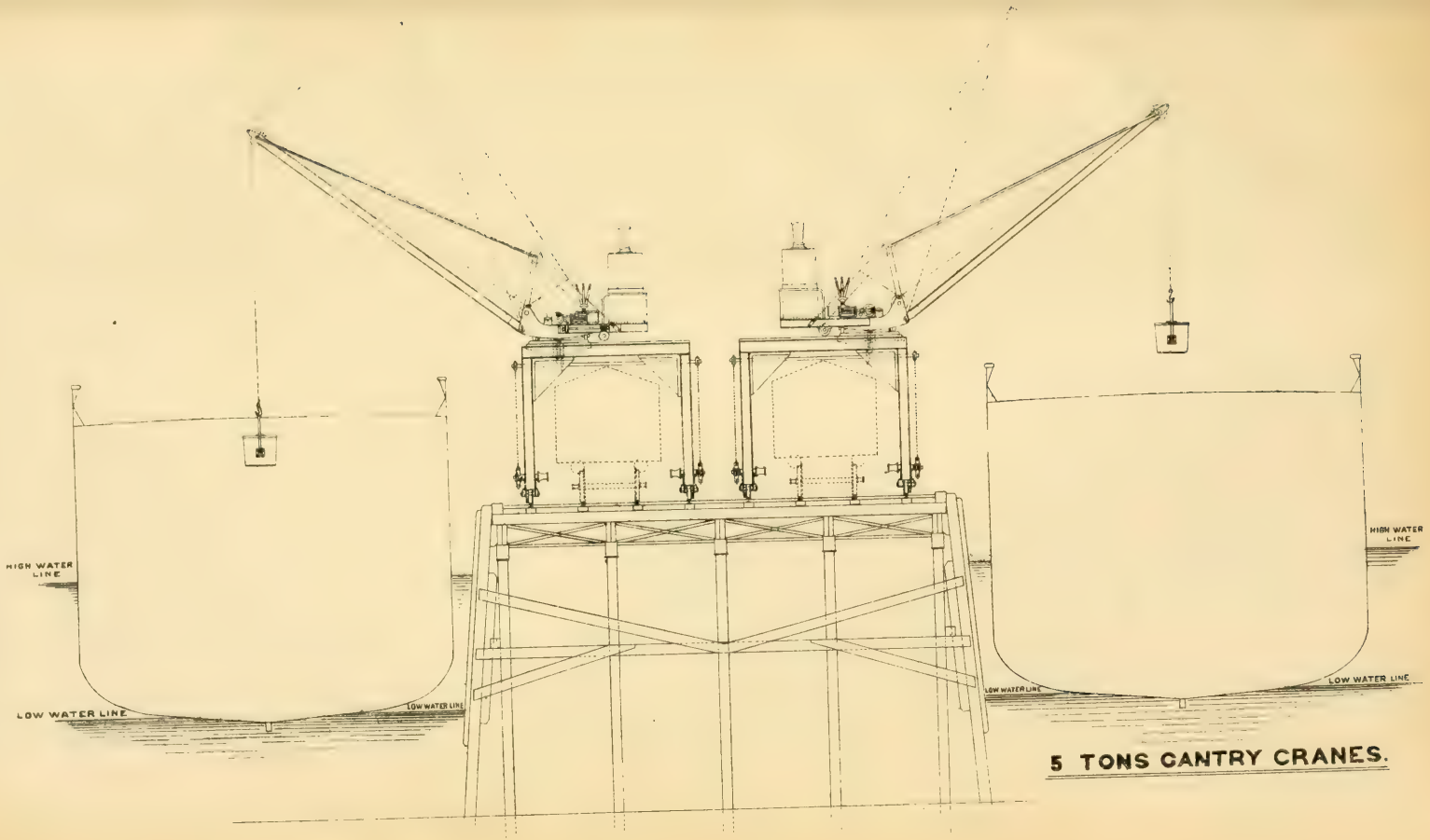


PLATE XX.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. No. 332.
APPLEBY ON
CRANES AS LABOR SAVERS.



5 TONS CANTRY CRANES.

PLATE XXI.
TRANS. AM. SOC. CIV. ENG'RS.
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CRANES AS LABOR SAVERS.

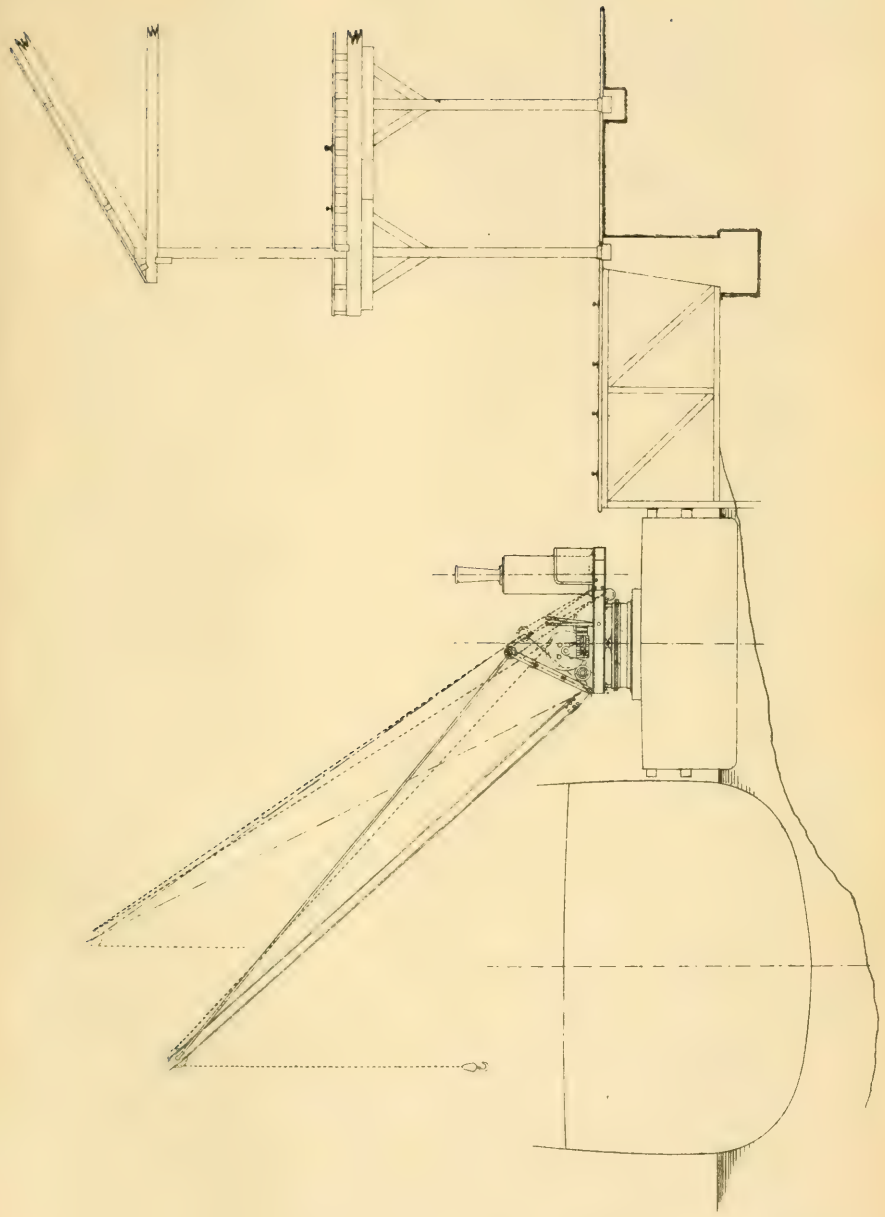

PLATE XXII.
TRANS. AM. SOC. CIV. ENG'RS.
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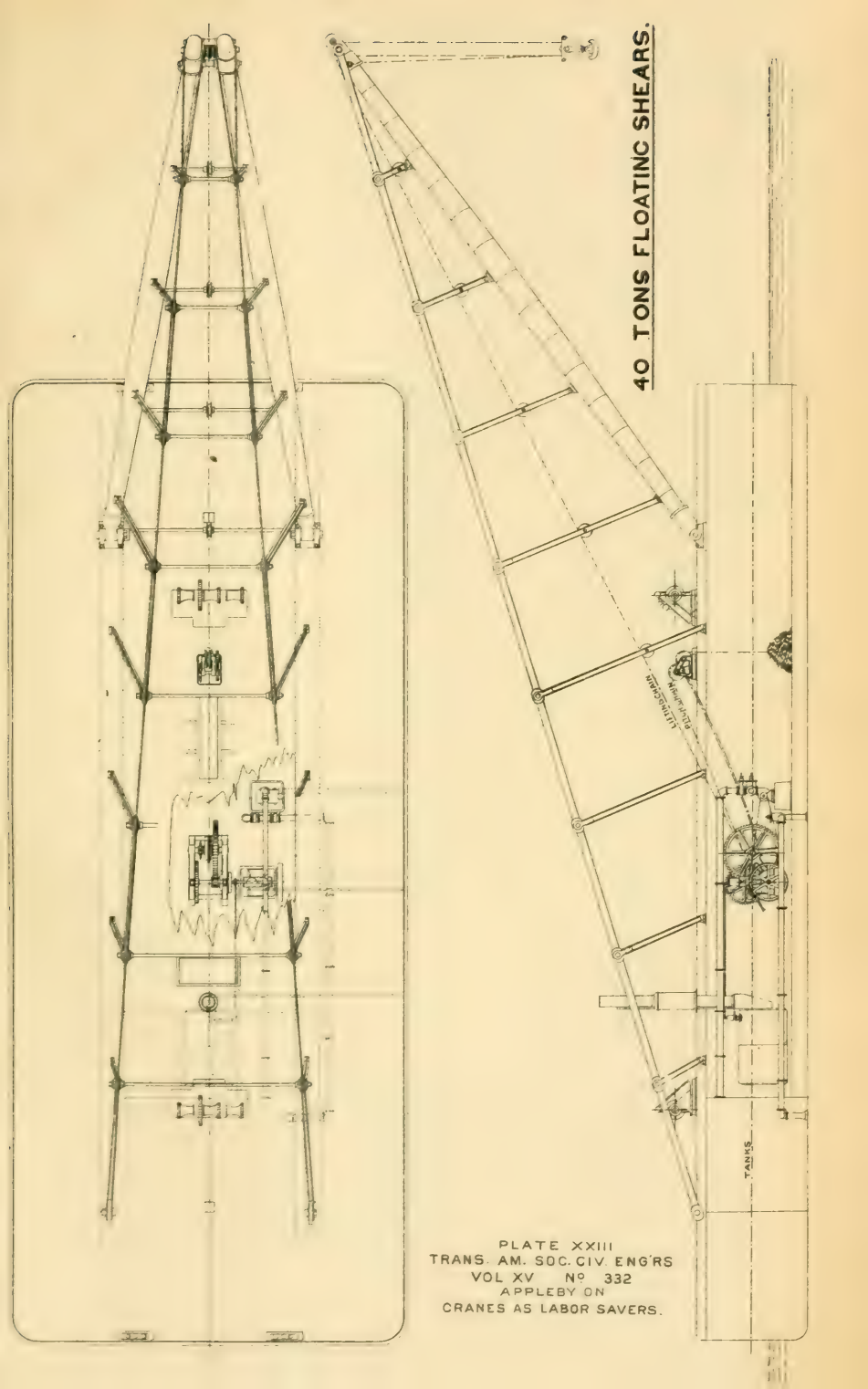

PLATE XXIII
TRANS. AM. SOC. CIV. ENG'RS
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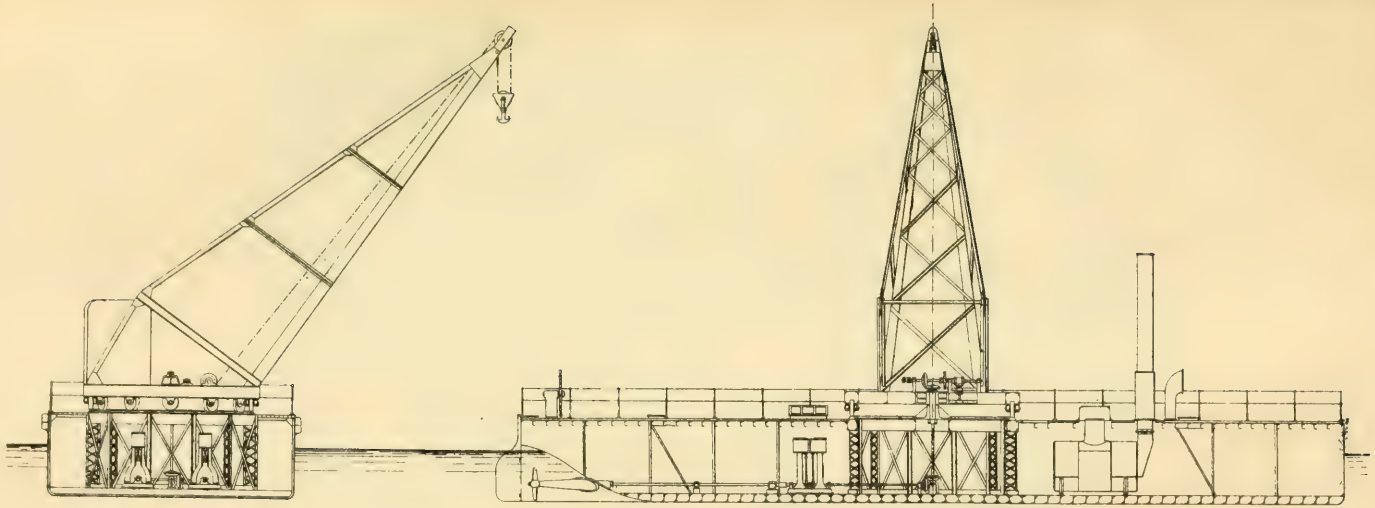
APPLEBY ON
CRANES AS LABOR SAVERS.



25 TONS FLOATING CRANE

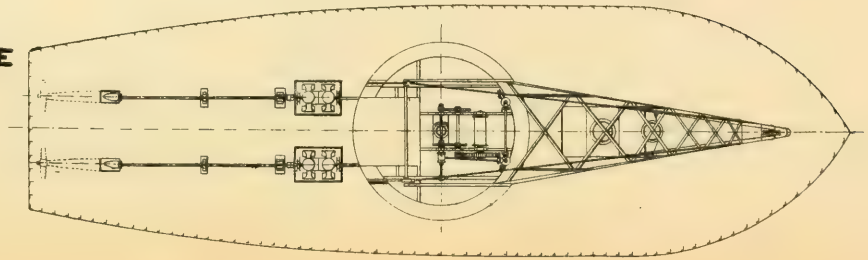


PLATE XXIV
TRANS. AM. SOC. CIV. ENG'RS.
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APPLEBY ON
CRANES AS LABOR SAVERS.

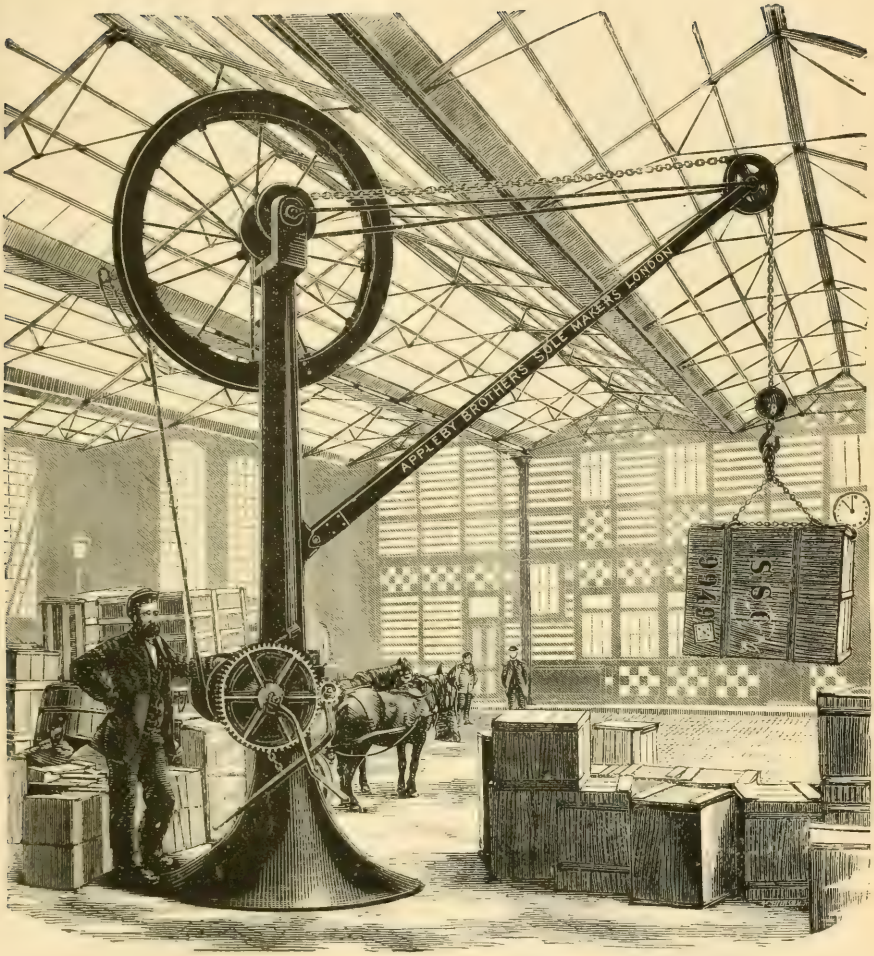

PLATE XXV
TRANS. AM. SOC. CIV. ENG'RS.
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APPLEBY ON
CRANES AS LABOR-SAVERS.

PLATE XXVI
TRANS. AM. SOC. CIV. ENG'RS.
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APPLEBY ON
CRANES AS LABOR SAVERS.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

333.
(Vol. XV.—May, 1886.)

THERMOMETER SCALES.

By FRED BROOKS, M. Am. Soc. C. E.

READ MARCH 3D, 1886.

Recent events make it desirable to give a little thought to the difference in the scales used to indicate temperature. It is well known that the scale in popular use in the United States marks the temperature of melting ice 32 degrees, and that of boiling water 212 degrees; while the scale of scientific men marks those temperatures 0 degrees and 100 degrees respectively. If engineering is a scientific profession, engineers may be expected to prefer the latter scale; but they actually use the former the most. The division of the interval between the fixed points into 100 parts has a superiority over that into 180, if there is any advantage in decimal reckoning. Those remarkable numbers, 212 and 32, have no peculiar thermometric virtue. It is unnecessary to draw a lengthy contrast between the two scales as to intrinsic merit. Charles T. Porter's standard "Treatise on the Richards' Steam Engine Indicator," says (p. 39 of third edition) "that the Fahrenheit scale begins in a blunder and ends nowhere." "If the Fahrenheit scale could be at once abolished, and the centigrade scale substituted in its place, we should

be rid of an arbitrary nuisance, which is already being banished from the domain of science, and is a real impediment to popular scientific advancement." This sentence suggests inquiry as to what hinders the desired substitution. The Fahrenheit scale cannot be instantaneously driven out of popular use; yet engineers, as they have occasionally to express temperatures, can do it in centigrade degrees, even though in some cases the equivalent Fahrenheit should be added in explanation. Will there ever be a better time than the present for taking this partial step toward the proposed change? Certainly this course is free from obstacles that sometimes stand in the way of similar improvements. For instance, it is regarded as an indispensable preliminary to such changes, to circulate abundant information in advance, and to provide conversion tables, double scales or diagrams. Well, for thermometers that is already done; the books that engineers use are full of tables and formulas for translating from one scale to another. Again, men do not buy and sell by the degree of temperature; but the thermometer is a scientific instrument, used mostly by intelligent people. There is not in this case that mass of ignorance to be instructed which there would be in the case of things like time-reckoning or coinage, with which everybody has to do.

National prejudices, though utterly irrelevant, too often interfere, as was noticeable in the recent adoption of a common prime-meridian. Happily, American views of the change now under discussion, cannot be distorted by such prejudices with regard to the countries where these thermometer scales originated—for Holland was the residence of Fahrenheit, who produced one of them in 1720; and Sweden, of Celsius, who devised the other in 1742. With those two countries we have the same grounds of sympathy. Sir Isaac Newton published in 1701 some results he obtained with a linseed-oil thermometer (see Philosophical Transactions No. 270, vii, which is at pp. 824–9 in Vol. 22); and Réaumur, the French naturalist, invented in 1731 a spirit thermometer with the scale which was used for many years in France and elsewhere. They both took 0 degrees to designate the temperature of melting ice, though they designated the boiling point differently from each other, and from either Celsius or Fahrenheit. Most branches of measurement are so connected, that a change cannot be made in one thing without some disturbance of others; thus time is related to angular measure, and coinage to weight. But temperature is independent; it has as little to do with one weight

or measure as with another, and people who differ widely in their preferences for a basis of weights and measures should be able to agree upon a thermometric scale. The centigrade scale is used in the meteorological service of Russia, a country which has for length measures the archine and the British foot; of Denmark, which has for length measures the alen and the Rhineland foot; and of the other continental countries of Europe, which have still another length measure. It is also used in places of such minor national rank as the Khedival Observatory in Egypt, and the National Observatory of Mexico. In England, the latest annual report, dated 27th July, 1885, from the Standards' Department of the Board of Trade, drew attention (p. 2) to the fact that various instruments, among which are thermometers, are not directly provided for by the Weights and Measures Act, so that at present the Standards' Department has no power to provide standards for them. So the Briton is not prevented by the law from using such a thermometer as he pleases; and the August, 1885, edition of the British Pharmacopœia, an official publication, contains centigrade temperatures throughout, in addition to Fahrenheit temperatures, which alone appeared in former editions. Likewise in the last decennial revision of the United States Pharmacopœia, published in 1882, both were inserted. The 309th paragraph of the "Revised Regulations for the Government of the Marine Hospital Service," approved 24th April, 1885, and authorized by Supervising Surgeon-General Hamilton, Secretary Manning and President Cleveland, is as follows:

"In recording thermometric observations, medical officers and acting assistant-surgeons of the Marine Hospital Service will make use of and refer to the centigrade scale only."

The United States will be among the last of the nations of high civilization to join in the use of the centigrade scale, even if she brings it into use with the utmost possible rapidity.

The subject of thermometer scales concerns engineers as well as other people, and in that respect resembles standard time, in which this Society has taken a very active interest; but there is the following difference: The centigrade scale can be used in engineering reports, experiments, etc., whenever they are made, be it sooner or later; no date for its adoption need be fixed in advance, as was required for simultaneously setting clocks and watches to the standard time.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

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334.
(Vol. XV.—May, 1886.)

THE STEAMSHIP AMERICA.

By ROBERT GORDON, M. Am. Soc. C. E.

READ JUNE 13TH, 1884.

This steamer was built by Messrs. I. & G. Thomson, of Clydebank, and was designed to form a new departure in transatlantic communication. The National Steamship Company, of Liverpool, has long been known for its safe, but somewhat slow, freight-carrying vessels, which have often been selected by the British Government for the transport of troops. The Company, profiting by the results of its own experience and a study of that of rival lines, came to the conclusion that it is necessary to separate fast passenger transport from that of freight in ocean ships, very much as is now done on railways, and determined to try a class of vessels which should be able to steam through the water at a rate of 18 or 19 knots per hour continuously through a long voyage. The aim was to insure a maximum of commodious passenger space and of speed, with a minimum of coal expenditure and of port dues.

These results have been so far attained, that the steamship America proved herself on her first voyage from Liverpool to New York and back, one of the fastest vessels afloat; and, for her capacity, one of the most economical in the use of coal. She was able to maintain a speed

of over 19 knots per hour continuously for many hours, while burning only 180 to 190 tons of coal per day; while other large vessels burn 300 to 340 tons per day without reaching that speed. A short notice of the principal factors in her design, on which these results depend, may be of some interest, and is respectfully offered to the Society, with the kind permission of her builders and owners.*

In the shape of her hull, the America is the embodiment of principles discovered by the late Mr. W. Froude, who, while experimenting on differently shaped models of ships, found, contrary to the expectation and practice of naval architects, that a comparatively short and broad ship, with fairly fine entrance and exceedingly fine run aft, carried on continuously curving lines throughout, could be driven through the water at a higher rate of speed with a less expenditure of power than a long and narrow ship of the same capacity with fine entrance and parallel-sided body. Probably the greater amount of skin friction in the latter vessel might account for this result, but the relation of the resistances arising from this, and from the generation of waves in deep water, to the shape of the ship at different speeds, still requires some elucidation. For many years naval designers had gone on increasing the length of vessels in proportion to their width, in the hope that with the same cross-section, greater carrying capacity could be attained without decreasing the speed or much increasing the driving power.

The steamship City of Rome,† for instance, is 600 feet long over all, with an extreme breadth of 52 feet 3 inches, and depth of hold of 37 feet. Her midship cross-section on 26-feet draught would be about 1 300 square feet. She is estimated to weigh 8 000 tons, with a total displacement on 26-feet draught of 13 500 tons, or carrying capacity of 5 500 tons. The America has 459 feet extreme length, and breadth of 51 feet 1 inch, and depth amidships of 38 feet 6 inches. On 26-feet draught her cross-section will be some 27 square feet less than that of the City of Rome—or nearly equal to it—while her total tonnage is 8 830, and carrying capacity 2 830 tons. The indicated horse-power of the former vessel is over 14 000, but, as is well-known, the power of her engines and boilers had to be very much increased over the original

* This information has been obtained from Mr. Alex. MacLennan, Superintending Engineer of the National Steamship Co.; and Mr. Dover, Chief Engineer of the steamship America.

† The steamship City of Rome is described according to the original designs in the Proceedings of the Institution of Mechanical Engineers for 1880, p. 336 *et seq.*

design, and she, even now, averages only 16 knots per hour on a coal expenditure of over 300 tons per day, while the America averages 18 knots on her consumption of 180 tons. The Oregon burnt 337 tons when keeping up the same speed.

The general arrangement and design of the America are clearly shown in the drawings given, so that it is only necessary to mention here some of the peculiar features in her construction.

Hull.—With a length between perpendiculars of 432 feet, and width of 51 feet 1 inch, she has a depth amidships to the upper deck of 38 feet 6 inches, and to the promenade deck of 47 feet. This promenade deck is 250 feet long, with spaces for hatches and winches at each end, but the turtle-back, poop and forecastle run on the same line at the bow and stern. If this deck were continuous throughout, or made so that the strains could be transferred by suitable struts and ties, the vessel might be considered as a girder, with upper and lower chords 47 feet apart; but as it is she is treated by the builders as a beam of but $38\frac{1}{2}$ feet depth, and they have used steel with a compressive strain of 32 to 36 tons per square inch above the neutral axis, and of 29 tons below it. The ship is of mild steel throughout, except in a few specified places, such as the promenade deck.

There are three decks, continuous throughout the vessel: the upper deck; the main deck at $29\frac{1}{2}$ feet, and the lower deck $21\frac{1}{2}$ feet above bottom of keel: while an orlop deck is brought in at $13\frac{1}{2}$ feet wherever permitted by the arrangement of the machinery. The frames of the ship are spaced 27 inches apart, and the flooring beams 54 inches apart, while two rows of stanchions are placed under each beam wherever practicable. The floor beams are of patent bulb-steel, varying from 11 inches to 9 inches, according to requirements; while the flooring plates vary from $\frac{1}{2}\frac{1}{4}$-inch steel to $\frac{1}{3}\frac{0}{2}$-inch. Extremely strong web frames and reverse frames are carried through the machinery rooms; and four strong keelsons on each side of the center keelson suitably stiffen the vessel longitudinally. The shell-plating varies in thickness from $\frac{8}{1}\frac{}{6}$-inch to $\frac{1}{1}\frac{3}{6}$-inch, according to position. All plates are accurately fitted to their position; the joints are covered with butt-straps, and throughout the greater part of the ship are treble riveted. Eleven water-tight bulkheads, carried to the upper deck, divide the ship into twelve compartments, the longest of which is not over 48 feet; and any two of these may be filled with water without sinking the vessel. The

bulkheads vary in thickness from $\frac{7}{16}$-inch to $\frac{5}{16}$-inch plate-steel, and are carried by strong frames. Three coal bunkers carry together 1 650 tons of coal. The ship is schooner-rigged.

Engines and Boilers.—The engines are direct acting, with one high-pressure cylinder of 63 inches diameter, and two low-pressure, each 91 inches diameter, with 66-inch stroke. Each cylinder has four piston valves, coupled two and two above and below; the high-pressure of 25 inch diameter 9-inch stroke; the low, 32-inch diameter, and 12-inch stroke. The steam enters from the boiler at 90 pounds pressure, and passes from the high-pressure cylinder to the reservoir at about 26 pounds pressure, whence it is taken through the low-pressure cylinders, expanding to the condensers, where a vacuum of 26 inches is easily maintained by the air-pumps and the action of two Gwynne's pumps, throwing from 4 000 to 5 000 tons of water each per hour through the condensers. These are worked by separate engines of about 40 indicated horse-power. The total tubular surface in the condensers is about 12 000 square feet. Two air-pumps, of 36-inch diameter and 33-inch stroke, are worked directly by the main engines. The estimated power of the engines is over 7 000 horse-power.

Six double-ended boilers, $17\frac{1}{2}$ feet long and about 14 feet 4 inches in diameter, produce steam at 90 pounds pressure, while an extra single ended boiler, 11 feet long and 14 feet 8 inches in diameter, furnishes it at 95 pounds pressure. The total grate-surface of the seven boilers is 883 square feet; the tubular surface nearly 21 000 square feet. The total weight of the seven boilers is about 333 tons. They are constructed of steel plating from $\frac{5}{16}$ to 1-inch thick. The furnaces are of corrugated steel. The tubes are about $3\frac{1}{2}$ inches external diameter. Each boiler carries three safety valves of $5\frac{3}{8}$ inches diameter, all inaccessible. Two oval-shaped funnels, each divided longitudinally by a diaphragm, rise 100 feet above the keel of the ship.

The engines are provided with Dunlap's governor, and special starting-gear, arranged so that the speed can be adjusted independently of the governor. It is found in practice that 65 to 66 revolutions of the engine give a speed of about 18 knots to the vessel through smooth water, while 68 to 70 carry her at about 19 knots.

The crank shafts are of steel. The propeller shaft is of forged scrap-iron, 36 feet long and 22 inches diameter. The thrust is carried by ten collars, with about 3 000 square inches of bearing. The pro-

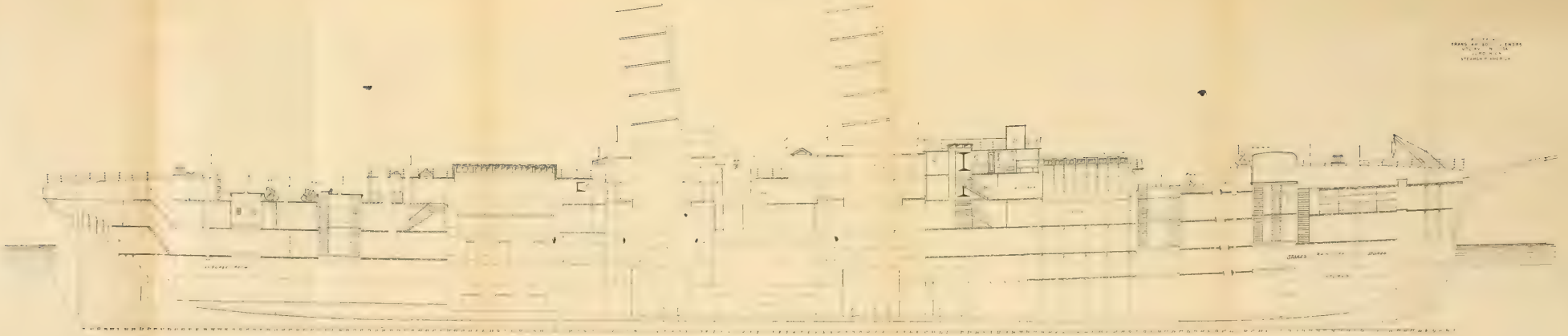
peller is four-bladed, 22 feet in diameter. The leading side has a regular pitch of 29 feet; the following side a pitch varying from 24 to 34 feet. The blades are of manganese bronze; the boss is of steel. The weight of the former is about 15 tons; of the latter, about 9 tons with the bolts. The total surface of the four blades is 164 square feet.

The vessel is designed to carry 670 cabin and 300 aft passengers. The state-rooms are very spacious, and each one, as well as the larger between-deck spaces, is connected to ventilating tubes which terminate in the chimneys, and secure a thorough renewal of fresh air. A complete system of drainage from the cabins, and decks and bilges in the several compartments, is made by piping and pumps, worked by the engines. The ship is lighted throughout by electricity, the Swan arc light being used. A special feature in the design is the saloon, which occupies the full breadth of the ship, and has a large central space rising through the upper deck. This is covered by a double dome, the interior of painted glass, the exterior of iron, with electric lamps between, which give a brilliant illumination. A gallery forms a music and conversation room at one end. The decorative work has been carried out with great taste, and the whole design reflects great credit on the builders.

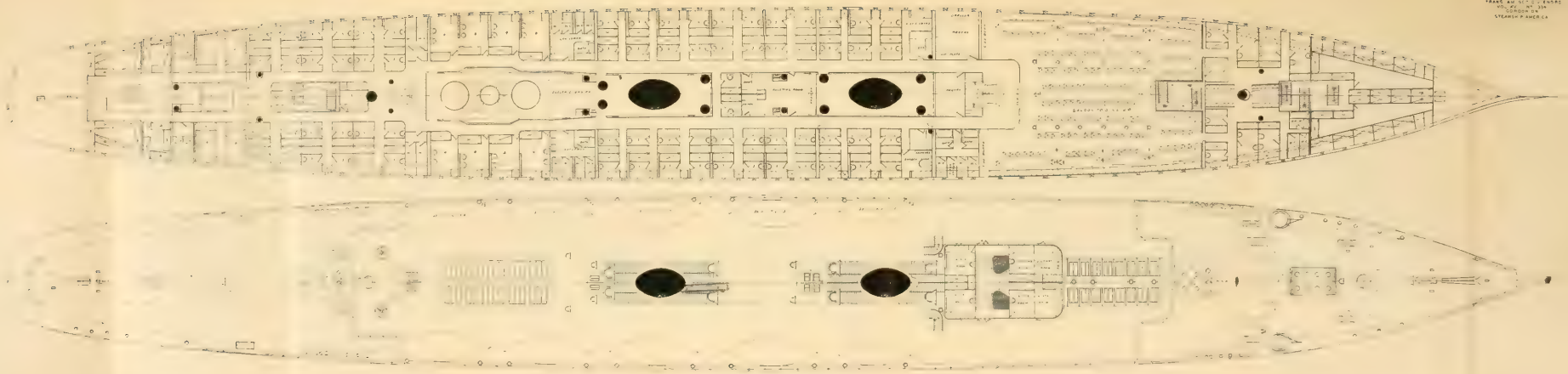
The following plates are appended:

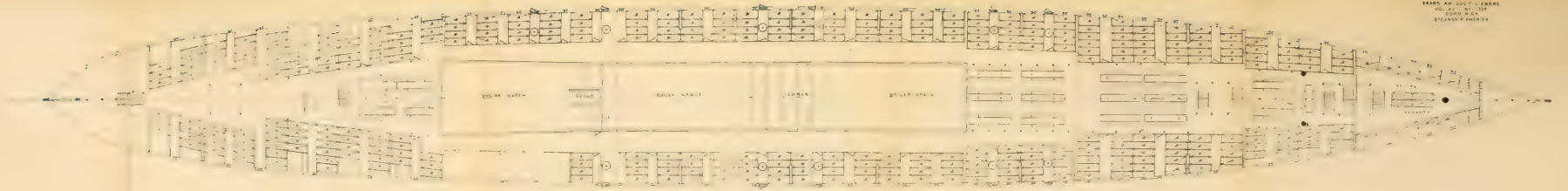
XXVII.	Steamship	America.	—Longitudinal Section.
XXVIII.	"	"	Plans of Main and Bridge Decks.
XXIX.	"	"	Plans of Upper and Lower Decks.
XXX.	"	"	Sections at Frames 19, 22, 29, 45.
XXXI.	"	"	Midship Section.
XXXII.	"	"	Propeller.
XXXIII.	"	"	Plan and Section showing engines.

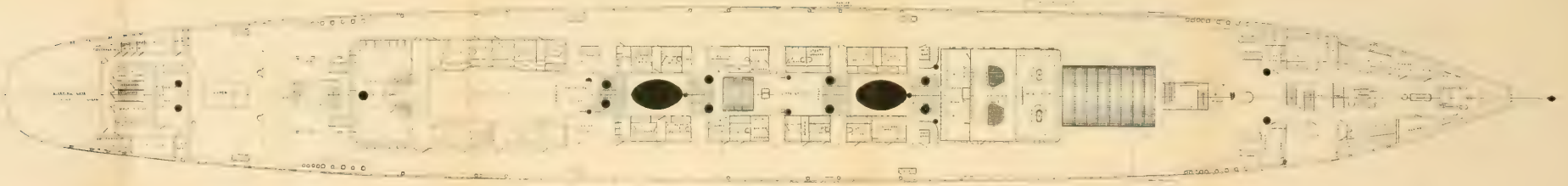
S.S. America.



Main Deck



Bridge Deck.

Lower Deck.



Upper Deck

PLATE XXII
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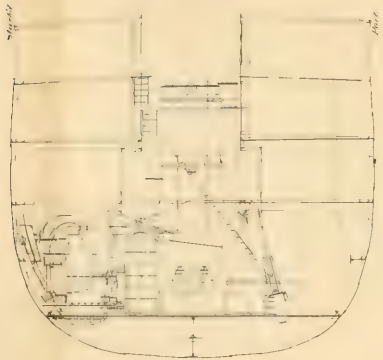
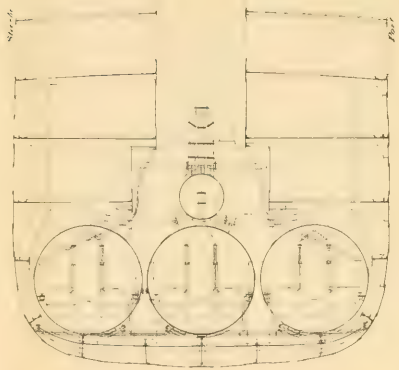
SCHOOK DR
STEAMSHIP AMERICA

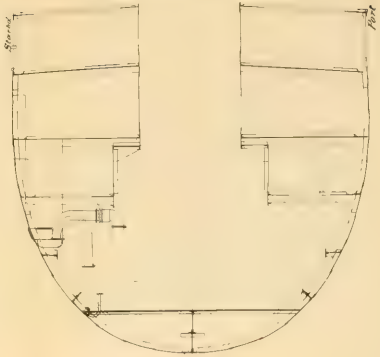
Section at Frame. № 29 looking aft.



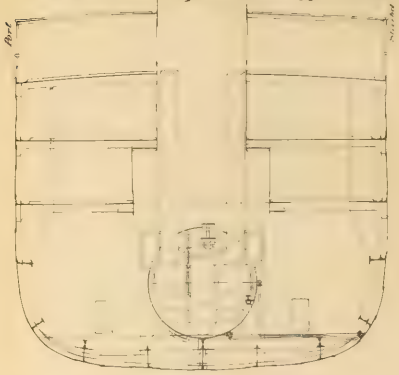
Section at Frame. № 19. fore body; looking aft.



Section at Frame. № 43 looking aft.



Section at Frame. № 22. fore body; looking forward.



Midship Section of S.S. America.

PLATE XXXI
TRANS. AM. SOC. CIV. ENG'RS
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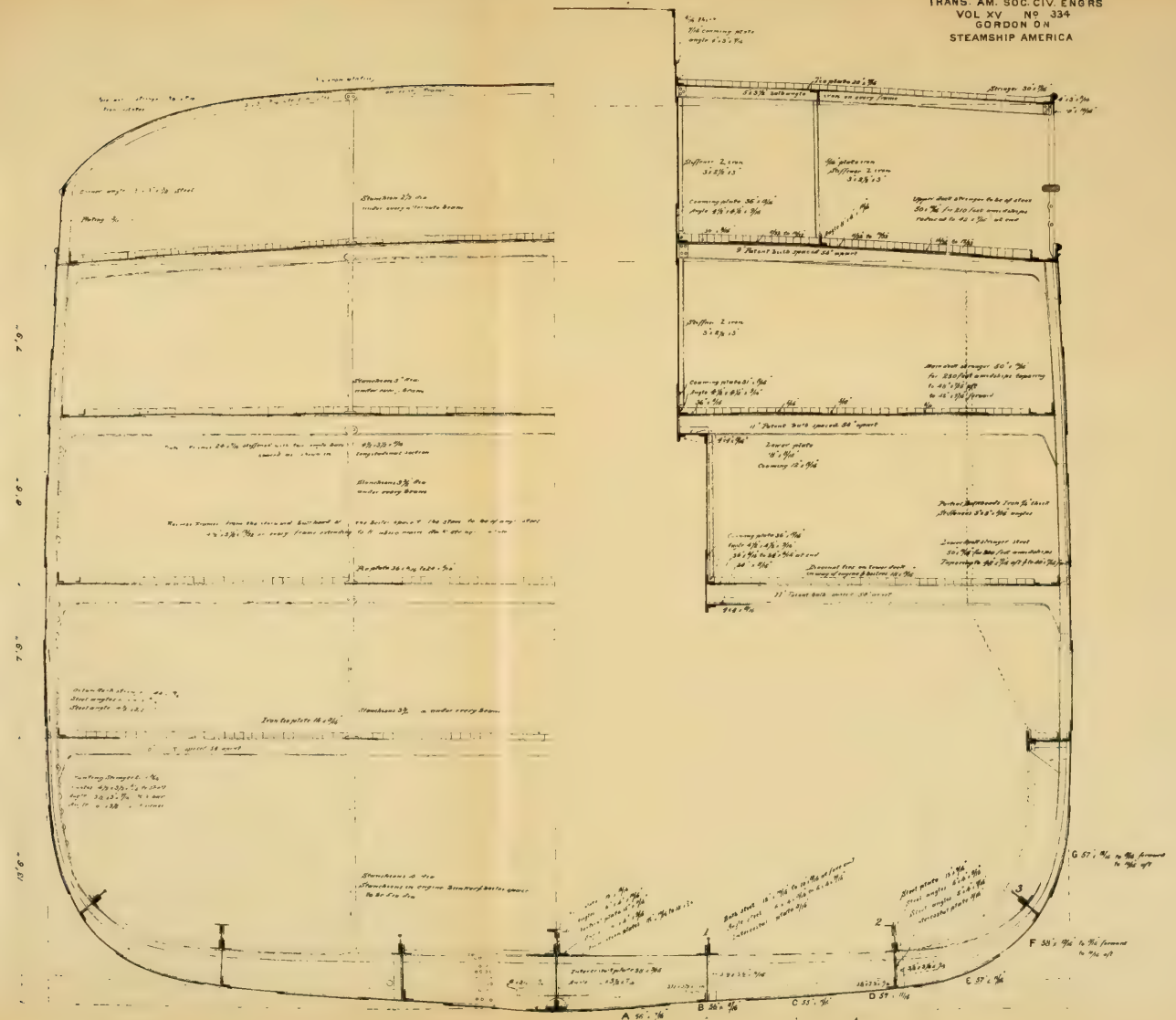
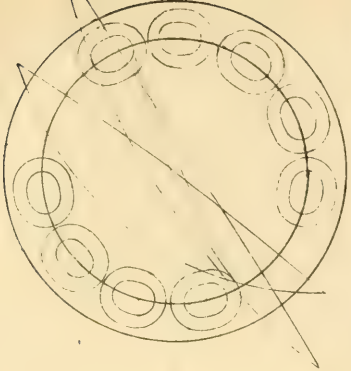
GORDON ON
STEAMSHIP AMERICA


Stringer 30"x 7/16
Upper deck stringer to be of steel 30"x 7/16 for 220 feet amidships reduced to 24"x 7/16 at ends
Upper deck stringer 30"x 7/16 for 220 feet amidships tapering to 40"x 7/16 aft to 42"x 7/16 forward
Lower deck stringer 30"x 7/16 for 220 feet amidships Tapering to 36"x 7/16 aft & forward
Bilge strake 18"x 7/16 to 15"x 7/16 at ends
Bilge strake 18"x 7/16 to 15"x 7/16 at ends
Keel plate 11"x 7/16
Garboard strake 6'-0"x 7/16
Intercostal plate 36"x 7/16
Intercostal plate 36"x 7/16
A 56"x 7/16
B 56"x 7/16
C 53"x 7/16
D 57"x 7/16
E 57"x 7/16
F 58"x 9/16 to 7/16 forward to 7/16 aft
G 57"x 9/16 to 7/16 forward to 7/16 aft



S.S. AMERICA

Size of Propeller 22 ft. per blade
Surface " " 184 sq ft
Uniform pitch on Locating circle 29
Aft. " " following " 24-34
Mean " " 29 ft.

PLATE XXXII
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GORDON ON
STEAMSHIP AMERICA

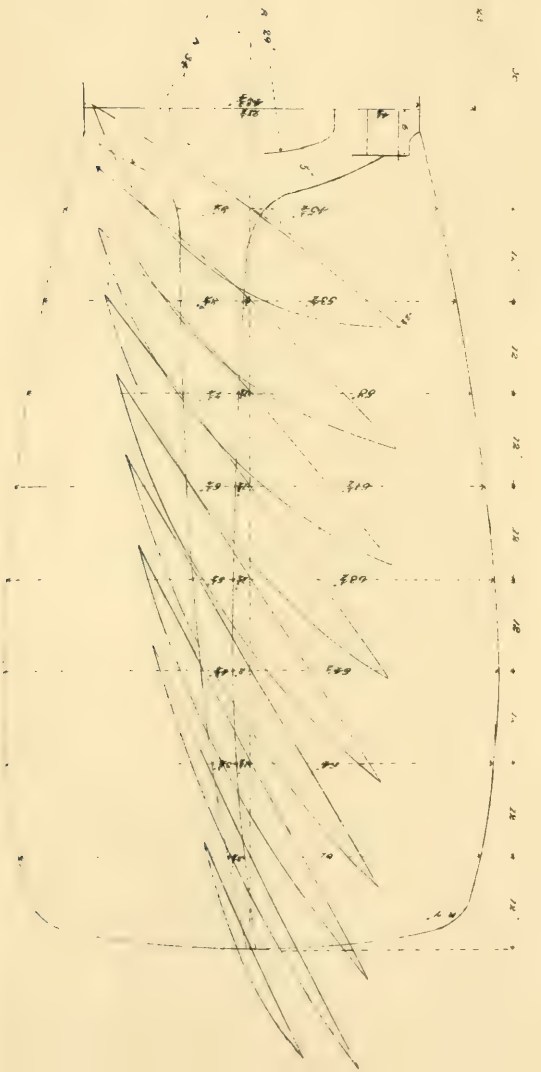
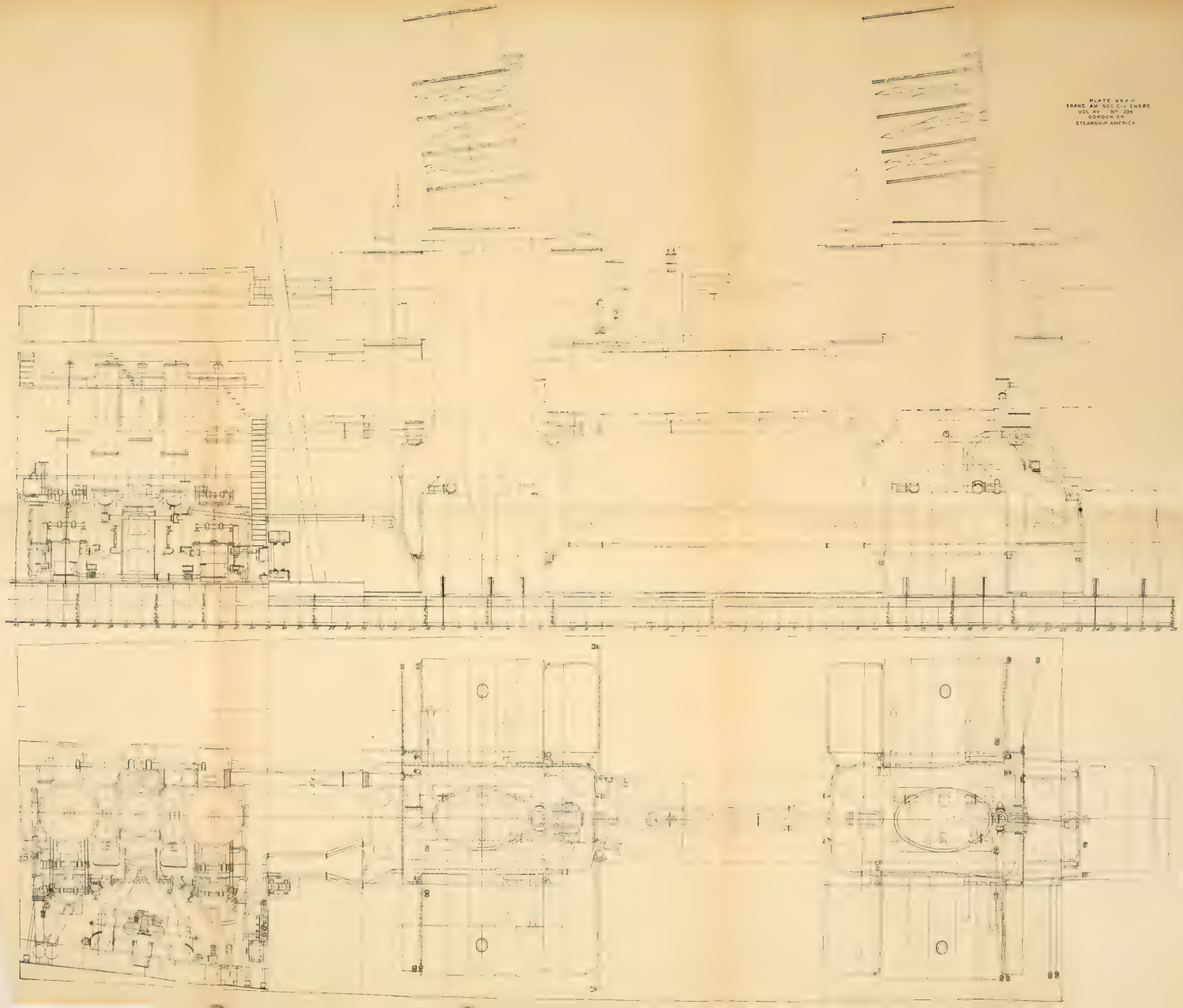


PLATE XXVII
TRANS AM SOC C-E ENGRS
VOL XV NO 334
GORDON ON
STEAMSHIP AMERICA



AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

335.

(Vol. XV.—June, 1886.)

ON SPECIFICATIONS FOR STRENGTH OF IRON BRIDGES.

By JOSEPH M. WILSON, M. Am. Soc. C. E.

READ AT THE ANNUAL CONVENTION, JUNE 25TH, 1885.

WITH DISCUSSION.

It has been within the province of the author, in his connection with the Pennsylvania Railroad Company, to prepare the specifications on which plans for all iron bridges for that line should be designed.

A long experience not only in designing, but also in the supervision and periodical inspection of bridges in actual use on the various lines of the company, has given the author opportunity of observing their action in service from year to year, and of noting any suggestions which might be furnished as to modifications or improvements in design. This experience has been of considerable value in forming the author's judgment on many points, and the results have necessitated more or less modification, from time to time, of the specifications under which work was designed. The great increase, however, in the weight of motive power and train service, and in the speed of running, has required of late years some very marked changes.

The specifications which are herewith submitted (see Appendix) are the result of theory combined with practice, and this paper is offered in

the hope that it may contribute something to the profession. These specifications are now standard for the Pennsylvania Railroad Company.

To meet the question of the continual increase in the weights of engines, necessitating constant changes in the assumed loading of bridges, it was decided several years ago to adopt for computation certain typical forms outlined under the author's direction, which, while covering the then existing service, would, it was believed, represent the advance for a longer period, and yet be reached in the near future. The loads from these typical consolidation and passenger engines are given in the specification, and results have already more than justified their adoption, as although no engines have as yet been built on the Pennsylvania Railroad with these precise weights, yet another type has actually developed in construction (P. R. R., Class M) which, in particular cases produces even heavier loading than that given by the typical engines, and indicates that the limit of increase has been by no means yet reached.

The specification starts out with the well-known requirement that the structure must be wholly of wrought-iron, and then follows with the live loads as represented by the typical and "Class M" engines, diagrams being given for each. The maximum, as given by either of these methods of loading, is required to be used, and in calculating web-members of trusses and girders, the cross-girder load under the drivers is to be considered as the head of the train, the load under the preceding cross-girder being neglected. This is on the side of safety.

The dead load to be carried is then stated, and its mode of distribution is given for the loaded and unloaded chords. The points from which general dimensions are to be taken for calculation are designated, and with the conditions now given the designer computes the stresses by one of the recognized methods.

The maximum and minimum stresses in compression and tension being found for the various members of the structure, depending upon the type of bridge selected by the designer, the specification then states the formulas according to which the permissible working stresses in these pieces are to be determined. These formulas are modified from those of Launhardt and Weyrauch as given by Weyrauch. ("Strength and Calculation of Dimensions of Iron and Steel Constructions." New York, D. Van Nostrand, 1877.)

Launhardt's formula requires:

For pieces subject to one kind of stress only (tension or compression):

$$a = u\left(1 + \frac{t-u}{u}\,\frac{\text{Min. } B}{\text{Max. } B}\right)$$

Weyrauch determined (for pieces subject to stresses acting in opposite directions):

$$a = u\left(1 - \frac{u-s}{u}\,\frac{\text{Max. } B'}{\text{Max. } B}\right)$$

where

a = ultimate working strength per unit of section under assumed conditions of loading.

u = ultimate strength per unit of section for any number of repetitions of load.

t = ultimate strength per unit of section for a single static load.

s = ultimate vibrating strength per unit of section when the opposite stresses are equal.

Max. B = the greatest stress upon the member, whether of tension or compression.

Min. B = the least stress of the same kind.

Max. B' = the greatest stress in the opposite sense.

Now,¶ assuming with Weyrauch that $t = 1\frac{1}{2}\,u$ and $s = \frac{1}{2}\,u$, there results:

$$a = u\left(1 + \frac{\text{Min. } B}{2\,\text{Max. } B}\right)$$

and

$$a = u\left(1 - \frac{\text{Max. } B'}{2\,\text{Max. } B}\right)$$

These formulas, with the proper substitution for u, depending upon the kind and quality of material used, will give the ultimate strength for the varying conditions of live and dead load quite satisfactorily, but they do not take account of the question of impact, etc., such as would be caused by the weight and motion of the train, irregularity of track, and the vertical pressure due to the inclination of the connecting rod of the engine, as well as centrifugal force developed from unbalanced weight in the wheels or other revolving parts. This question has been more or less discussed by engineers for a long time. Mr. B. Baker, C. E., in his work on "Long and Short Span Railway Bridges" (London, 1873), treats of the subject, and to provide for this, as well as the varying

conditions of live and dead loads, suggests the addition of a percentage to the usual working stress for bridges of short spans. This method of meeting the question has been adopted by quite a number of engineers. The following list, showing the percentages of increase given to static stresses by various designers, has been compiled from a number of examples of specifications to which the author has had access.

Edge Moor Iron Company :

Span in feet.....	20	30	40	50	60	70	80
Per cent.....	30	25	20	15	12	10	8

Keystone Bridge Company, for truss members only :

Span in feet.....	30	45	60	75	90	105	120	135	150
Per cent.....	20	15	12	10	8	6	4	2	0

For stringers, floor-beams and counter-ties, 25 per cent.; floor-beam hangers, 100 per cent.; riveted connections of stringers and floor-beams, 50 per cent.; middle-ties and posts, 10 per cent.

Mr. G. Bouscaren, M. Am. Soc. C. E.:

Span in feet.....	30	45	60	75
Per cent.....	25	20	15	10

Floor-beam hangers, 50 per cent.

Mr. Mace Moulton, M. Am. Soc. C. E., founded on B. Baker's deductions :

Span in feet....	10	20	30	45	60	80	100
Per cent.....	40	35	30	25	20	15	10

Mr. C. Shaler Smith, M. Am. Soc. C. E.:

Middle-ties, 20 per cent.; counters, 30 per cent.; end suspenders, 20 per cent.; long suspenders, 20 per cent.; floor-beam hangers, 50 per cent.; track-girders, 30 per cent.; floor-beams, 20 per cent.

Henderson bridge specifications :

Span in feet.....	30 to 45	45 to 60	60 to 75	75 to 120
Per cent. compression..	20	15	10	—
" tension.....	40	25	20	15

Floor-beam hangers and track-girder connections, 100 per cent.; vertical suspenders, 50 per cent.; track-girders, floor-beams, counters, trestle-posts and compression members of less than 30 feet span, 25 per cent.

These authorities adopt only one value of static stress for all classes of material, and in all cases, except for the Henderson bridge, 10 000 pounds per square inch is used for tension, and 8 000 pounds per square inch

for compression. The Henderson bridge specifications required 10 000 pounds per square inch for tension, except in track-girders and floor-beams, for which 8 000 pounds was used. For compression, 8 000 pounds was adopted, as given in the other cases.

Prof. William Cain, A. M., C. E. ("Maximum Stresses in Framed Bridges," New York, 1878), takes a more rational method of solving the problem. He assumes that as impact is due to and increases with the action of the live load on a member, its effect will vary inversely with $\frac{\text{Min. } B}{\text{Max. } B}$ and he writes empirically for the formula of Launhardt, $b = \frac{u}{n}\left(1 + \frac{\text{Min. } B}{\text{Max. } B}\right)$, n being a factor of safety; and he takes the value of $\frac{u}{n}$ for all iron at 7 500 pounds per square inch. Here the value of b is the permissible working stress per unit of section. This is equivalent to making the value of $t = 2\ u$ in Launhardt's formula previously given. Adopting this relationship between t and u, and still retaining that of $s = \frac{1}{2}\ u$, the original formulas become

$$a = u\left(1 + \frac{\text{Min. } B}{\text{Max. } B}\right) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$a = u\left(1 - \frac{\text{Max. } B'}{2\ \text{Max. } B}\right) \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

These formulas, it must be remembered, are for ultimate strength. If for u, working values are assumed instead of ultimate, the value of a becomes permissible working stress per unit of section, and adopting this arrangement, the author believes that the results of practical experience are most rationally met.

The value of u is taken by the author for double-rolled iron in tension (links or rods), at 7 500 pounds per square inch; for rolled iron in tension (plates or shapes), at 7 000 pounds per square inch; and for rolled iron in compression, at 6 500 pounds per square inch.

For Min. B = Max. B, $a = 2\ u = t = 15\ 000$ pounds, corresponding to all dead load, and for Min. $B = 0$, $a = u = 7\ 500$, corresponding to all live load.

Plate XXXIV, accompanying this paper, gives by diagram the value of a for various values of $\frac{\text{Min. } B}{\text{Max. } B}$ according to the formulas of the specification.

For high test iron in tension, the line $A\,A$ should be used for the

first case, pieces subjected to one kind of stress only, and the line $A\,D$ for the second case, pieces subject to stresses acting in opposite directions. For ordinary iron in tension, use the line $B\,B$ for first case, and $B\,E$ for second case. For ordinary iron in compression, use the lines $C\,C$ and $C\,F$.

On the same diagram there will be found plotted the values of a resulting from the various methods of percentage as used by Edge Moor Iron Company, Keystone Bridge Company, etc., previously quoted, and although the lines shown may be somewhat in error, as the author was obliged to assume his own values of loads for the various spans, still the results are valuable as a matter of comparison with those obtained by the formulas.

The permissible stress a being found, that for members in compression must be reduced in proportion to the ratio of the length to the least radius of gyration of the section.

The author has for many years used Rankine's formulas for this purpose. Late experiments, however, made on a large scale, and the investigations of a number of authorities on the subject, do not altogether agree with the results of these formulas.

During the years 1875 to 1879, inclusive, Mr. G. Bouscaren, M. Am. Soc. C. E., made a series of experiments on the strength of wrought-iron columns, the results of which will be found in a paper contributed by him to the Society (Trans. Am. Soc. C. E., Vol. IX, p. 447). From these results it would appear that between Rankine's and Gordon's formulas, the former is the more accurate of the two : "that iron of the highest modulus does not necessarily make the strongest column; that it is essential in built columns to thoroughly fasten together the several parts or segments that they may act in a solidary manner as one continuous metallic body, and to guard not only against lateral flexure or buckling of each part, but also against longitudinal relative motion ; that a due regard to the economy of material demands that the thickness of the metal, and in built columns the spacing of the rivets, be so proportioned that the column shall fail by flexure as a whole before any local buckling or flexure shall take place ;" that Rankine's formula with the constant $\frac{1}{3000}$ for columns with flat ends, assumed at double its value for columns hinged on pins at both ends, "is practically correct."

The tables accompanying Mr. Bouscaren's paper give the results of his experiments on various columns of different kinds, forms of cross-sections, etc., and made by different manufacturers.

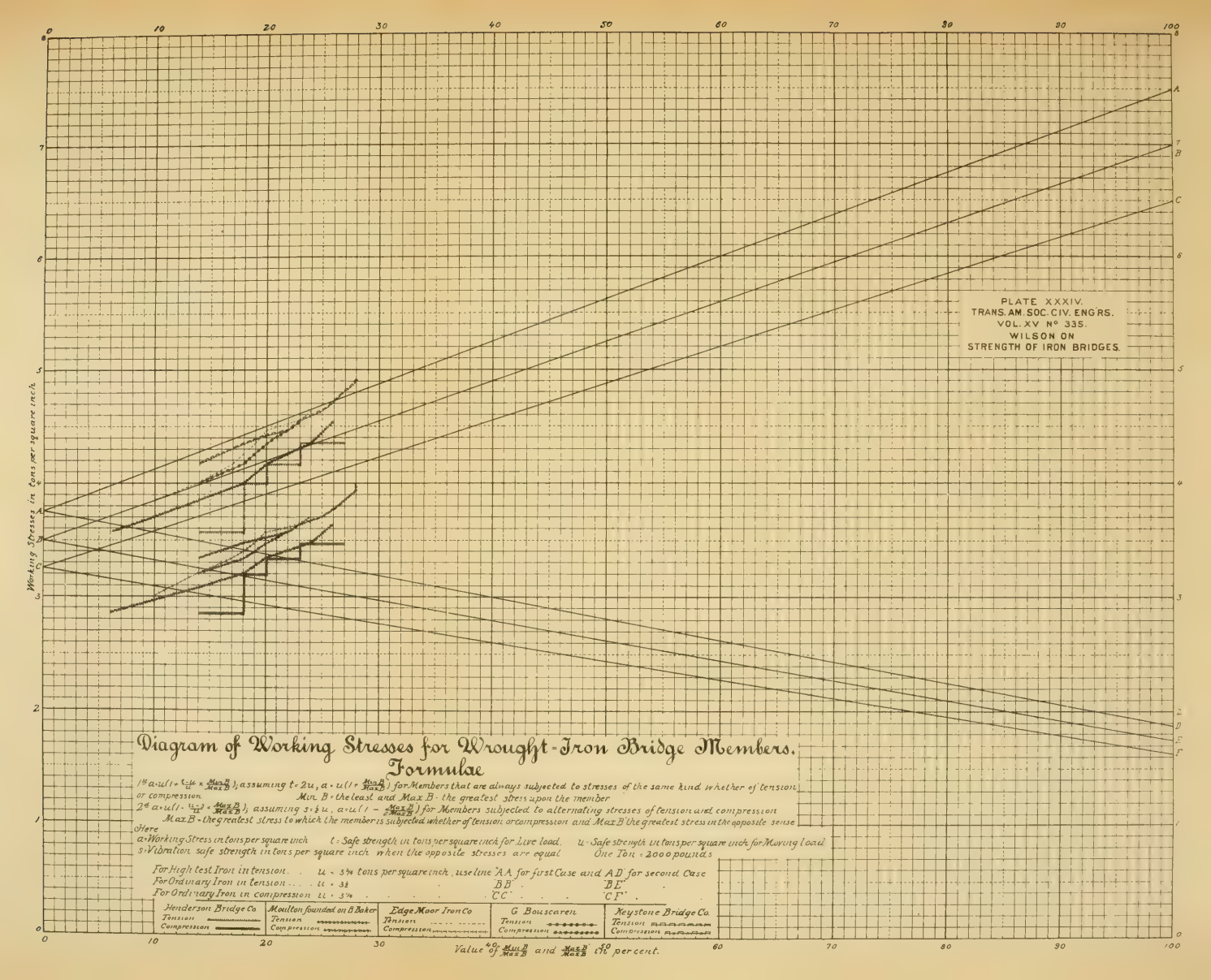


PLATE XXXIV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV N° 335.
WILSON ON
STRENGTH OF IRON BRIDGES.
Working Stresses in tons per square inch.
Value of Min B / Max B and Max B / Max B to 100 per cent.
A
B
C
D
E
F
Diagram of Working Stresses for Wrought-Iron Bridge Members.
Formulae
1st a = u(1 + t/u × Min B / Max B); assuming t = 2u, a = u(1 + Min B / Max B) for Members that are always subjected to stresses of the same kind whether of tension or compression
Min B = the least and Max B = the greatest stress upon the member
2d a = u(1 − 1/12 × Max B / Max B); assuming s = ½u, a = u(1 − Max B / 2 Max B) for Members subjected to alternating stresses of tension and compression
Max B = the greatest stress to which the member is subjected whether of tension or compression and Max B the greatest stress in the opposite sense
Here
a = Working Stress in tons per square inch
t = Safe strength in tons per square inch for Live load.
u = Safe strength in tons per square inch for Moving load
s = Vibration safe strength in tons per square inch when the opposite stresses are equal
One Ton = 2000 pounds
For High test Iron in tension . . u = 3¾ tons per square inch, use line 'AA' for first Case and 'AD' for second Case
For Ordinary Iron in tension . . . u = 3½ . . 'BB' . . 'BE'
For Ordinary Iron in compression u = 3¼ . . 'CC' . . 'CF'
Henderson Bridge Co
Tension
Compression
Moulton founded on B Baker
Tension
Compression
Edge Moor Iron Co
Tension
Compression
G Bouscaren
Tension
Compression
Keystone Bridge Co.
Tension
Compression

In November, 1879, Messrs. Clarke, Reeves & Co., Mems. Am. Soc. C. E., had a series of experiments made upon a number of Phœnix wrought-iron columns at Watertown Arsenal, Mass., and the results of their work were furnished to the Society in 1882 (Trans. Am. Soc. C. E., Vol. XI). These experiments on breaking strength, elastic limit, etc., embraced twenty columns of 8.04 inches diameter, formed of four rolled arc segments having flanged riveted connections, and lengths varying from 8 inches to 28 feet, the sectional area of each being slightly in excess of 12 square inches; also two columns 11.8 inches in diameter and 18.3 square inches sectional area, formed of six segments and having lengths of 8 feet 9.5 inches and 25 feet 2.65 inches respectively. The results demonstrated the inaccuracy of Gordon's formulas for hollow columns, as might be supposed, and indicated the advisability of using separate formulas for long and short columns, as suggested by Hodgkinson some years ago.

Rankine's formulas corresponded much more closely with the experiments, indicating, however, that the value of $\frac{1}{36000}$ for a was too large, $\frac{1}{100000}$ being better.

In the discussion on this paper, Mr. Theodore Cooper, M. Am. Soc. C. E., proposes a series of formulas for columns of different forms of section and for the various conditions of square, pin or round-ended. He thinks that the allowed working strain should be a proportion of the elastic limit of the material, not of the rupturing or crippling-point. Instead of computing the crippling strength of a column, and taking, say one-fourth, one-fifth or one-sixth of the result as the allowed strain, he prefers changing the formulas so as to give directly the allowed strain, and suggests modifications for this purpose.

Mr. James Christie, M. Am. Soc. C. E., read before the Society, in 1883, a very complete paper on experiments made by him at the Pencoyd Iron-works for the purpose of determining the comparative resistance to compression of long and short struts of rolled angle, tee-beam and channel sections. This paper forms a valuable contribution to available data on the practical resistance of columns.

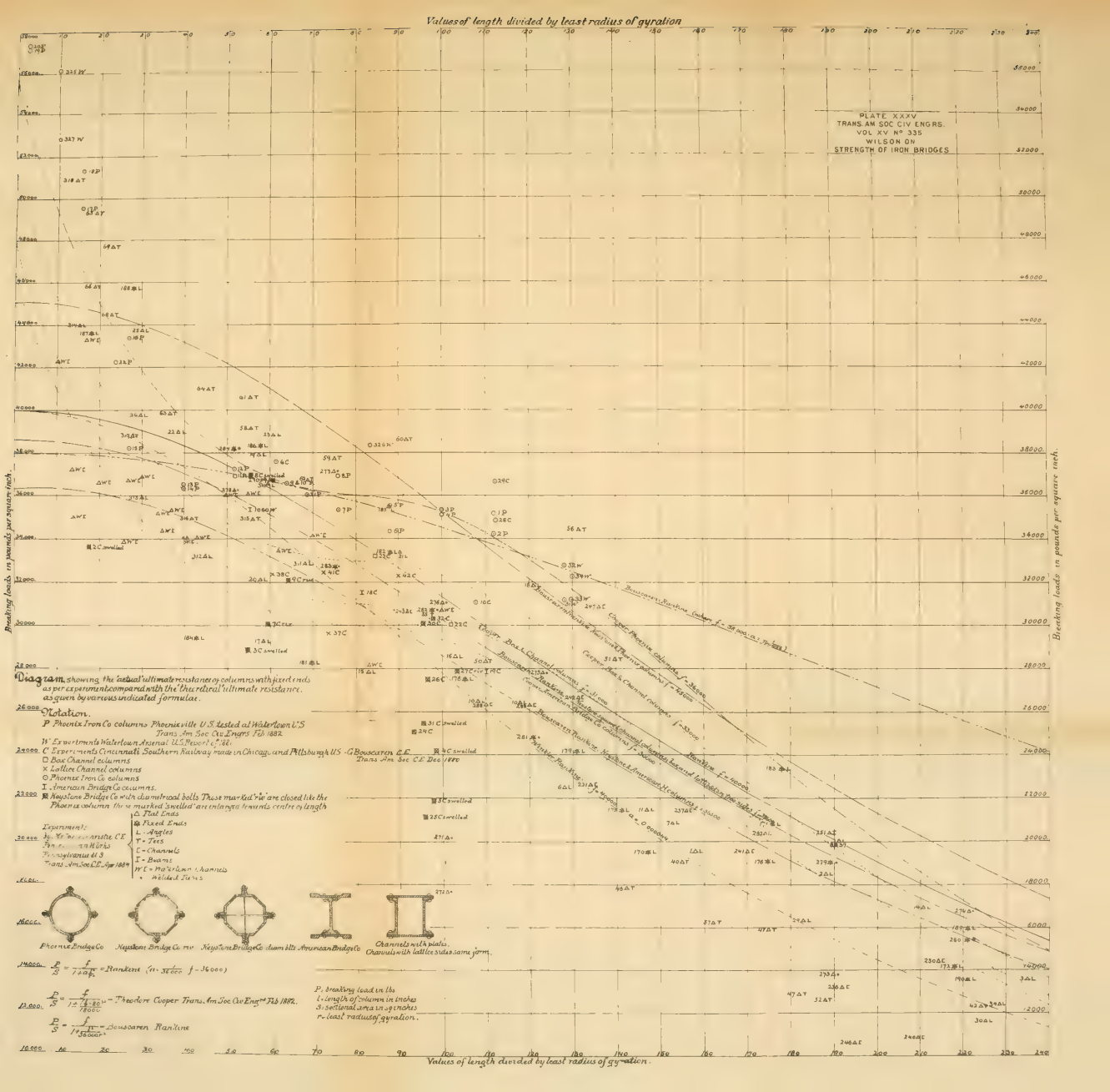
On Plates XXXV and XXXVI accompanying, will be found the actual ultimate resistance, as shown by the principal experiments reported in these various papers, Plate XXXV being for fixed, and Plate XXXVI for pinned ends. On Plate XXXVI are shown also the results of four experiments made under direction of the author at

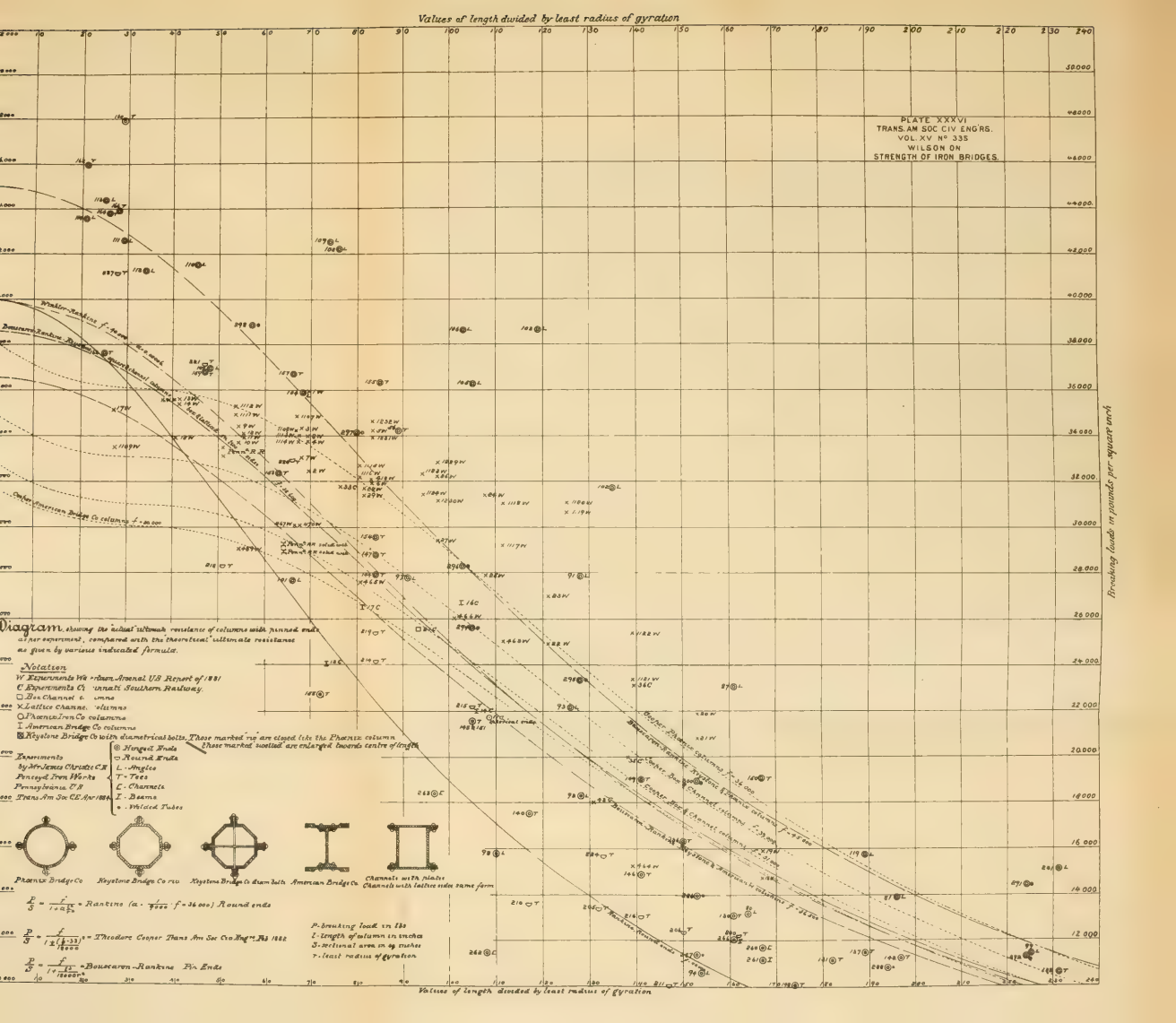
Watertown Arsenal several years ago. They are designated on the plate Penna. R. R. Two of these were made on columns built of channels with a middle web of plate secured by angles, and two upon columns of channels having their flanges connected by cross-plates and lattice-bars. All the specimens were of the same size and length of channels, and had pins at both ends, proper shoes being made to fit against the plates of the testing machine and to receive the pins. The columns were tested with the pins vertical, and no effort was made to counterbalance the weight of the specimens.

The results of the various experiments are plotted in the diagram by using for abscissæ the values of the length divided by the least radius of gyration or $\frac{l}{r}$, and for ordinates the breaking stress per unit of section. The scales adopted have been so taken as to avoid crowding the results together, and to enable a better comparison to be made between them. Very often in such diagrams the vertical scale is taken too small.

On these same plates will be found also plotted the lines of ultimate resistance for columns as by the formulas of Rankine, Cooper and Bouscaren-Rankine, the latter being modifications of Rankine's formulas as proposed by Mr. Bouscaren ("Report on the Progress of Work and Cost of Completing and Maintaining the Cincinnati Southern Railway," G. Bouscaren, January 1, 1880, p. 20). Mr. Bouscaren takes the ultimate resistance of column per square inch $= R = \frac{f}{1 + \frac{l^2}{36\,000\, r^2}}$ for flat ends, and $R = \frac{f}{1 + \frac{l^2}{18\,000 r^2}}$ for pin ends, and the value of f being assumed at 45 000 for Phœnix post, 38 600 for square post closed on all sides, or open and latticed on two opposite sides, and 36 500 for H post. The factor of safety he takes at one-fifth. There will also be found plotted Mr. Bouscaren's proposed modification of Rankine, using $f = 38\,000$, and $a = \frac{1}{100\,000}$ (Trans. Am. Soc. C. E., Vol. XI, p. 65), and Dr. Winkler's modification of Rankine (Proc. Inst. C. E., Vol. LIII, p. 302), where $f = 40\,000$; $a = 0.000\,044$. By examination of these diagrams the variation between the results of the formulas and the actual experiments may readily be observed.

After considerable consideration of the subject, the author has adopt-


Values of length divided by least radius of gyration
Breaking loads in pounds per square inch.
PLATE XXXV
TRANS. AM. SOC. CIV. ENGRS.
VOL. XV. No. 335
WILSON ON
STRENGTH OF IRON BRIDGES
Diagram showing the actual ultimate resistance of columns with fixed ends as per experiment compared with the theoretical ultimate resistance as given by various indicated formulæ.
Notation.
P Phoenix Iron Co columns Phoenixville U.S. tested at Watertown U.S. Trans. Am. Soc. Civ. Engrs. Feb. 1882.
W. Experiments Watertown Arsenal U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway made in Chicago and Pittsburgh U.S. - G. Bouscaren C.E. Trans. Am. Soc. C.E. Dec. 1880
□ Box Channel columns
× Lattice Channel columns
⊙ Phoenix Iron Co. columns
I. American Bridge Co columns.
⊠ Keystone Bridge Co with diametrical bolts. Those marked "w" are closed like the Phoenix column those marked "5 milled" are enlarged towards centre of length
Experiments:
by Kirkaldy, Hurst, Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884
Pinned Ends
Fixed Ends
L - Angles
T - Tees
C - Channels
I - Beams
W.L - Watertown Channels
Welded Tubes
Phoenix Bridge Co
Keystone Bridge Co rev.
Keystone Bridge Co diam. bolts
American Bridge Co
Channels with plates. Channels with lattice sides same form.
Rankine $\frac{P}{S} = \frac{f}{1+\frac{l^2}{36000r^2}}$ = Rankine (f = 36000 f = 36000)
Theodore Cooper Trans. Am. Soc. C.E. Feb. 1894. $\frac{P}{S} = \frac{f}{1+\frac{l^2}{18000r^2}}$
Bouscaren Rankine $\frac{P}{S} = \frac{f}{1+\frac{l^2}{36000r^2}}$
P. breaking load in lbs
l. length of column in inches
S. sectional area in square inches
r. least radius of gyration.
Values of length divided by least radius of gyration.


Values of length divided by least radius of gyration
PLATE XXXVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. N° 335
WILSON ON
STRENGTH OF IRON BRIDGES.
Breaking loads in pounds per square inch
Values of length divided by least radius of gyration
Diagram. showing the actual ultimate resistance of columns with pinned ends as per experiment, compared with the theoretical ultimate resistance as given by various indicated formulæ.
Notation
W Experiments Watertown Arsenal U.S. Report of 1881
C Experiments Cincinnati Southern Railway.
□ Box Channel columns
X Lattice Channel columns
○ Phoenix Iron Co columns
I American Bridge Co columns
⊠ Keystone Bridge Co with diametrical bolts. These marked 'no' are closed like the Phoenix column. These marked 'eccentric' are enlarged towards centre of length.
◉ Hinged Ends
○ Round Ends
L - Angles
T - Tees
C - Channels
I - Beams
• - Welded Tubes
Experiments by Mr James Christie C.E. Pencoyd Iron Works Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884
Phœnix Bridge Co
Keystone Bridge Co rivets
Keystone Bridge Co diam. bolts
American Bridge Co.
Channels with Th. plates
Channels with lattice sides same form
Rankine (a = 1/9000) f = 36000 Round ends
Theodore Cooper Trans. Am. Soc. C.E. Aug. 1882
Bouscaren-Rankine Fin Ends
P = breaking load in lbs
l = length of column in inches
S = sectional area in sq. inches
r = least radius of gyration

ed the following formulas (Nos. 3, 4 and 5 of the specification) for the allowable working stress in compression per square inch of section.

For both ends fixed:

$$b = \frac{a}{1 + \frac{l^2}{36\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For one end hinged:

$$b = \frac{a}{1 + \frac{l^2}{24\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

For both ends hinged:

$$b = \frac{a}{1 + \frac{l^2}{18\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where a = permissible stress previously found.

b = allowable working stress per square inch.

l = length of piece in inches, center to center of connections.

r = least radius of gyration of the section in inches.

These formulas are identical with those designated here as Bouscaren-Rankine, except that they are adapted to finding directly the working compressive stress from a value previously determined, which may be termed the permissible stress per square inch for a column of one diameter, or more correctly perhaps, of a length equal zero.

Plate XXXVII shows diagrams of the working stresses for pin-ended columns as determined from the formulas of Cooper, Bouscaren-Rankine and Weyrauch, plotted with values of $\frac{l}{r}$ for abscissæ and the working stresses per square inch for ordinates. The same diagram shows the plotted curve of working stresses by the author's formula, equation (5), the value of a being taken from eq. (1) with $\frac{\text{Min.}}{\text{Max.}} = 0.31$ very nearly, b for $\frac{l}{r} = 0$ becoming 8 500 pounds. This curve will of course be found identical with that of Bouscaren-Rankine, when the ultimate stress for short prisms is taken at 42 500 pounds, and the factor of safety, as specified by Mr. Bouscaren, at 5.

Pieces used in compression, which are continuous over points of support, are generally free, so that they may deflect from the action of

the compressive force in one direction in one panel of the structure and in the other direction in adjacent panels, thus producing all the effects of hinging at the ends. The specifications therefore provide that such pieces, unless so firmly fixed as to be incapable of such bending, shall be considered as hinged at the ends.

The author places great importance upon having the lines of the neutral axes of all members which connect together meet at the same point; and, if there is a pin connection, the pin should also occupy as nearly as possible this same position. After providing for this, the specifications go on to say how the upper chords of deck bridges shall be proportioned when the floor system rests directly on them, and states that all other members subject to direct stress in addition to the bending moment shall be similarly calculated. In this connection it may be stated that the average maximum effect by the transverse load from an engine on the upper chord considered as a continuous beam, will be found to be about three-fourths what it would be if the beam was supported only, and the moment is generally from a single concentrated load, although this depends altogether upon the length of panel.

The specification now follows with the requirements for the sizes of eyes on all pin-connected tensile members, and these proportions are considered quite sufficient when the best known methods of manufacture are used. In some cases, however, larger heads might be found necessary, and this is covered by a clause under "Quality of Material," which specifies that all links and rods, if tested to the breaking, shall part through the body and not through the head or pin-hole. It is possible that the future may see very considerable reduction from the proportions as specified, when better methods of manufacture are devised. Indeed, the Edge Moor Iron Co., of Wilmington, Delaware is now working eye-bars in both steel and iron by a new method of upsetting, without buckling or welding, which has given astonishing results. That company furnishes these bars with fifty per cent. excess in the width of head over bar when the diameter of the pin is less than the width of bar, and forty per cent. when the diameter of the pin exceeds the width of bar (really reversing the rule of the present specification when the diameter of the pin exceeds width of bar), and states that it will guarantee that the eyes will develop the full value of the bar even at only forty per cent. excess in head for the former case, and thirty-three per cent. for the latter. Experiments have been made at Edge

PLATE XXXVII
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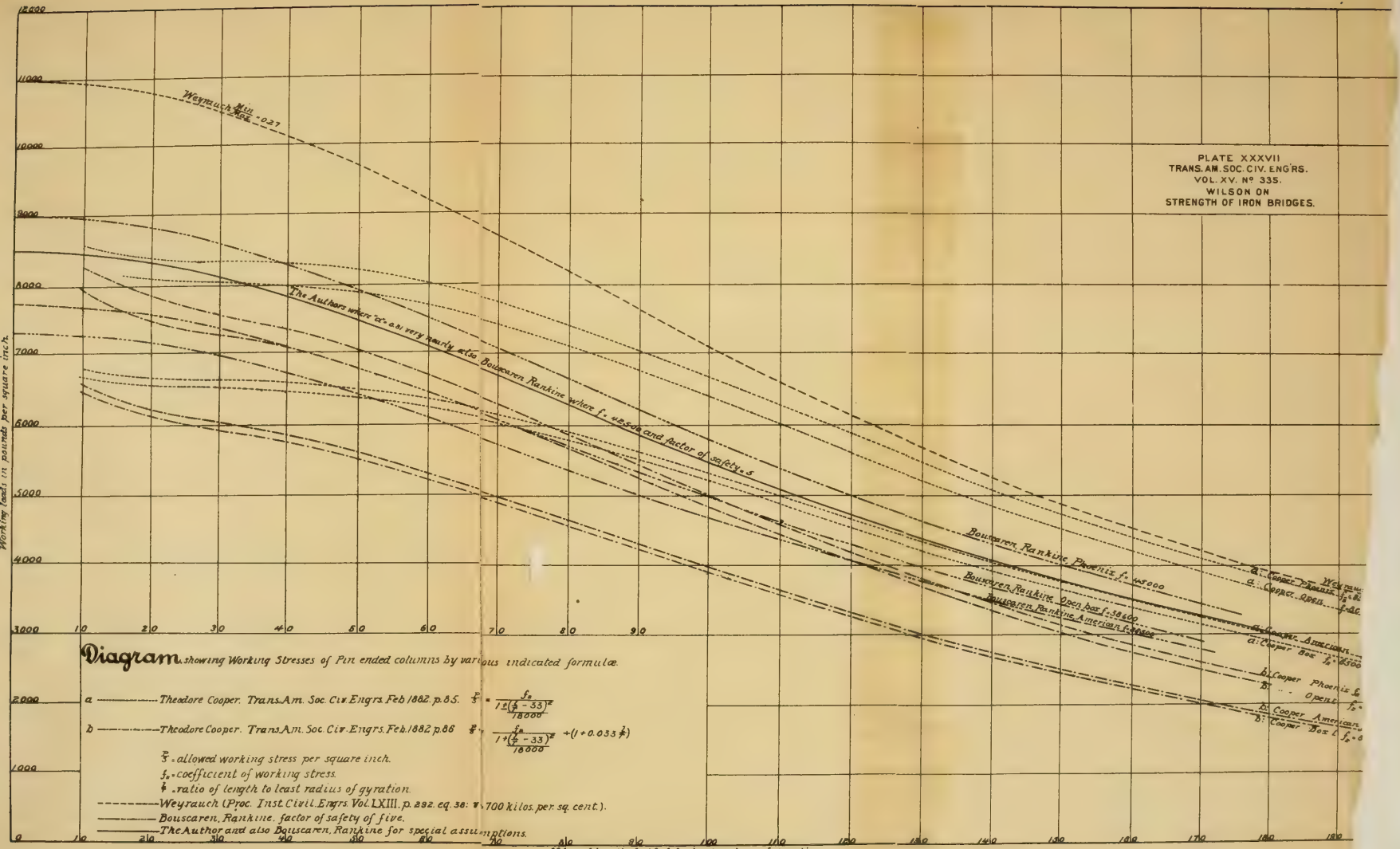


Diagram showing Working Stresses of Pin ended columns by various indicated formulæ.

a ——— Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882. p. 85. $\frac{P}{S} = \frac{f_o}{1 + \frac{1}{18000}\left(\frac{l}{r} - 33\right)^2}$

b ——— Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882. p. 86 $\frac{P}{S} = \frac{f_o}{1 + \frac{1}{18000}\left(\frac{l}{r} - 33\right)^2} + (1 + 0.033\,\frac{l}{r})$

$\frac{P}{S}$ = allowed working stress per square inch.
f_o = coefficient of working stress.
$\frac{l}{r}$ = ratio of length to least radius of gyration.
---------- Weyrauch (Proc. Inst. Civil. Engrs. Vol. LXIII., p. 282. eq. 36: f_o = 700 kilos. per sq. cent.).
——————— Bouscaren, Rankine. factor of safety of five.
——————— The Author and also Bouscaren, Rankine for special assumptions.

Moor on bars of this mode of manufacture having an excess in width of head as low as 20 per cent., and the bar has still parted through the body under the ultimate stress.

The table on page 400 is a record of some tests which have been recently made. The minimum limit has evidently not yet been reached.

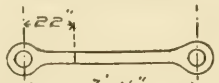
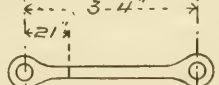
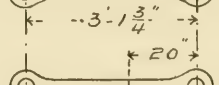
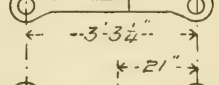
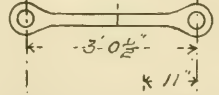
The rule given for the minimum diametrical bearing between pins and pin-holes is an old one, and experience has proved it satisfactory. It is preferred not having pins less in diameter than three-fourths the width of the bar.

The author considers it very essential that the pins be so proportioned and the pieces connected by them so packed, that the maximum stress per square inch on the outer fibers of the pins from the cumulative bending moments shall not exceed one and a half times the maximum tensile stress a in the members connected. It is true that many old bridges are yet in service where the pins would not by any means come up to this requirement of the specification, but there is no doubt that the stresses in the links are very unevenly distributed, and some of the pieces may be seriously overstrained. In such bridges it is very difficult to determine whether the results are dangerous, but they must certainly produce an effect of deterioration on the structure which may end seriously.

Among the tests already referred to as made for the author at Watertown Arsenal, was one on pin-connected links, to ascertain the effect of the bending of the pin on the links which it connected. It is hardly necessary at the present time to give a detailed account of this test, but it may be said to have fully confirmed the views already expressed.

In arranging the spacing of rivets, it will be noticed that the various details, etc., are to be constructed so that the compression members in these detailed parts are good for a working stress as high as would be obtained in a column of twelve diameters. In limiting the distance of rivets apart at right angles to the line of stress, at not more than thirty times the thickness of the thinnest external plate, it is desired to prevent any liability to buckling from stresses, which would otherwise be perfectly legitimate on the members. Stoney ("Theory of Strains," London, 1869, p. 210), in referring to the results of Mr. Hodgkinson's experiments on compression of wrought-iron in rectangular tubes, where the yielding is by buckling, not flexure, leads to the conclusion that the

REPORT OF TESTS ON STEEL EYE-BARS MANUFACTURED BY EDGE MOOR IRON CO., FROM BESSEMER STEEL MADE IN PENNSYLVANIA.

Date of Test, 1885.	Mark.	Length of Bar and Location of Break. Eye A.	Eye B.	Percentage of Excess in Heads. A.	B.	Size of Bar.	Area in Square Inches.	Diameter of Eye. A.	B.	Diameter of Pin. A.	B.	Limit of Elasticity in pounds per square inch.	Percentage of Elongation in length of 8 inches.	Percentage of Reduction of Area.	Strength in pounds per square inch.	Elongation of Pin-hole. A.	B.	Remarks as to Fracture, etc.
April 22..	No. 2			24.1	36.6	5" x1 1/16"	5.15	12"	12"	5 15/16"	5 7/16"	38 800	34.3	52.3	67 600	0.5"	0.14"	Silvery cupped with some granular spots.
" 23..	" 10			21.4	20.2	4" x1"	4.	10"	10"	5 3/16"	5 3/16"	37 000	37.5	58.2	61 100	1.51"	0.81"	Silvery cupped.
" " ..	" 11			26.5	26.5	"	4.	10"	10"	4 15/16"	4 15/16"	38 900	31.2	59.8	63 000	0.82"	1.36"	" "
" " ..	" 13			26.1	26.	5" x1"	4.85	11 1/2"	11 1/2"	5 7/16"	5 7/16"	41 200	40.6	55.1	61 100	0.79"	1.88"	" "
" " ..	" 15			23.5	23.3	4" x1"	4.	10"	9 3/8"	5 1/16"	4 7/16"	38 900	31.2	44.2	60 400	1.41"	0.94"	2/3 Silky, 1/3 Granular

thickness of plate for proper resistance should be not less than one-thirtieth of the breadth of the tube, or in the present application, of the distance between the lines of rivets by which the parts are assembled together. Where straps and plates are used merely for stiffening, and do not take part in the compressive resistance of the column, their thickness is limited to not less than one-fortieth, and for double straps, one-fiftieth of the distance between the rivets connecting them to the compressed members. There is very little positive data on which to base these rules for widths of plates, beyond practical experience and professional opinion. It is hoped that further experiments with built compression members will develop more reliable information. It has been claimed by some designers that by omitting the area of the connecting plate in arranging the required section of a column, this plate may be made thinner than if it was included. In reply to this it may be said that so long as this plate is continuous, it is not possible to omit it; that its apparent omission only increases the actual section of the column by that much, and thereby lessens the amount of stress a certain small proportion over the whole of it, the connecting plate included, but the latter still takes its proportion of strain.

As to the proper distance for rivets from the sides and ends of pieces, the specification, while cautionary, does not give any definite rule, but the author himself follows the rules of Edge Moor Iron Company. Detailed rules are given for lattice strapping and terminal cross-bracing plates, as determined by custom and experience.

The shear on the net section of any member is limited to the compressive working stress a, and for rivets twenty per cent. extra section must be allowed to cover inaccuracy in workmanship, etc. This agrees with general practice. It is required that no allowance shall be made for the web in calculating the flange sections of plate girders. The author is aware that the web is often included, and in the case of solid rolled beams the specification allows it. In built girders, however, which generally have thin webs, the author considers such practice as working very closely, the gain from it being very small and the material so used being really put in the structure for another purpose.

The succeeding clauses of the specification concerning transverse stress, also the placing of rivets, etc., in plate girders, require little or no explanation. In reference to the proper method of proportioning the web and stiffeners of plate girders there seems to be a difference of

opinion among authorities, or at least a want of clearness in explanation, and a failure to give the subject its most economical consideration, at least in theory. Professor Rankine gives a rule for proportioning the web, in which he considers the shearing stress at the neutral axis as equivalent to a pull and a thrust of equal intensity inclined in opposite ways at 45 degrees from the vertical, the web tending to give way by buckling under the thrust. The vertical shear being resolved into these two components, one of tension and the other of compression on the web, the latter is computed to resist the compressive force as a fixed column of rectangular section, with a length equal to the distance along a line inclined at 45 degrees between two of its vertical stiffening ribs; or, if it has no such ribs, between the upper and lower horizontal flanges, a factor of safety of six being employed. When ribs are introduced, Professor Rankine is hardly as clear as he might be. He considers these either as suspending pieces (ties) or pillars (struts), according to the position of the load either on the lower or upper flange, and appears to proportion them for the local loading, those at the ends of the girder being the only ones specified to take the entire proportions of the load which rests at those points, or in other words, the vertical shear. The ribs act by their stiffness to prevent buckling of the web, and in that sense, if placed sufficiently close together, by reducing the length of column of the web, lessen its required thickness. Otherwise the thickness of web is computed in the same way, with or without ribs. Mr. Stephenson seems to have had very much the same opinion as expressed by Professor Rankine, and Professor Airy in his investigations on the subject also had the same views, that the girder acted as a lattice truss of an infinite number of intersections, these making up the web, the material of which could sustain, without injury, forces of tension and compression acting at right angles to each other, the least resistance being to the compression.

So long as no stiffeners are used, this rule of Professor Rankine's appears rational and practical, giving satisfactory results. When, however, the girder becomes too deep for an economical and practical thickness of web, stiffeners are required, and on the above theory they must be close enough together to limit the ratio of length of column to thickness to an economical figure. A girder of 60 feet length and 5 feet depth of web for example, sustaining a load of 3 000 pounds per foot lineal, would require the stiffeners about 14 inches apart for a three-eighths web.

The author in his early experience, twenty-five years ago, was very much troubled by such results when he knew of girders of the above dimensions and loading, in actual use, having webs of only $\frac{3}{16}$ of an inch in thickness and stiffeners at intervals of 5 feet, these girders bearing up well under use. Some of them, now thirty years old, are still in service, although it is not claimed that they would come within the present specification. The author soon discovered, however, by means of a paper model with very thin flexible web, that when stiffeners were properly introduced the web no longer resisted by compression, but by tension, the stiffeners taking up the duty of compressive resistance, like the posts of a Pratt truss, and dividing the girder into panels equivalent to those of an open truss, the web in each panel acting as an inclined tie. Working on this theory, results were obtained that quite agreed with practical examples.

Stoney, in his "Theory of Strains" (London, 1869, p. 319), approaches nearer to the true elucidation of the problem, and appears to understand the matter, although he is hardly definite and positive enough as to the course that should be pursued.

Mr. Theodore Cooper, M. Am. Soc. C. E., in an article on the rules for pitch of rivets and thickness of web in riveted plate girders (*Van Nostrand's Engineering Magazine*, Vol. XVII, p. 209), gives a rule very similar in its operations to that of Prof. Rankine, but in which he uses a factor of safety of only one-fourth, and for the extreme limit allows a length of column corresponding to a working resistance of 3 000 pounds per square inch.

While Mr. Cooper appears to introduce his vertical stiffeners properly for the whole shearing force, he would require for the web a much greater thickness, or would, it seems to the author, be obliged to introduce stiffeners considerably closer than practical experience would justify. Thus, for the example previously quoted, of a girder 60 feet span and 60 inches deep, the thickness of web required to resist buckling would be, on his rule, $\frac{3}{4}$ of an inch, or to use a $\frac{3}{8}$-inch web, the author presumes Mr. Cooper would place stiffeners $2\frac{1}{2}$ feet apart. Believing, as the author does, that the web resists by tension and not by compression, he would be perfectly satisfied that this would be safe enough, but if he was obliged to consider the effect as compressive, he would not. If the limit of depth of web is taken at eighty times the thickness, the limit of the diagonal in which the force, if compressive,

would really act, and which is the length of the column of resistance, would be about 113. It is the fact of the web actually resisting by tension which saves the case.

The theory of the author is that the flanges, with a portion of the web, act to carry the load and to concentrate it at the vertical stiffeners, if it is not already there. The latter act as the vertical columns in a Pratt truss would do, carrying the shear, which is resolved on to the web in tension in a diagonal direction, panel by panel, towards the abutment. The conditions of loading will control the frequency of struts, as there is, of course, a limit to the ability of the flange to carry intermediate loads to the struts, and this should receive due consideration.

The requirements in reference to number, size, spacing, bearing, etc., of rivets, exercise so important a control over the thickness of web, that after it is determined for these, it will probably be found quite sufficient for anything that will otherwise come upon it, the stiffeners, of course, being properly placed.

The requirements of the specification for wind strains and the proper proportioning of the lateral system are, it is believed, in accordance with the latest American practice in this respect.

Great care has been taken in framing the conditions for Quality of Material and Workmanship to insure having what is desired and what the present advanced state of manufacture can produce, without asking for what it is impossible to obtain except at an extraordinary cost. A practical experience of some years and extended intercourse with those who are foremost in the manufacturing interests of our country have, of course, formed the basis for this portion of the specification.

APPENDIX.

Specifications for Strength of Iron Bridges.

The structure to be wholly of wrought-iron, and designed to carry the following live loads on each track, headed in the same direction on adjacent tracks.

First.—Two typical consolidation engines coupled, weighing, with tender, 86 tons each, distributed as shown in figure 1, page 406, connected with a train weighing 3 000 pounds per foot lineal of track; or

Second.—Two typical passenger engines, coupled, weighing, with tender, 88 tons each, distributed as shown in figure 2, page 406, and connected with a train weighing 3 000 pounds per foot lineal of track; or

Third.—One class M engine, weighing, with tender, 66½ tons, distributed as shown in figure 3, page 406, and connected with a train of 3 000 pounds per foot lineal of track. The maximum given by either of these methods of loading is to be used in proportioning every member of the structure.

In calculating web-members of trusses and girders, the cross-girder load under the drivers is to be considered as the head of the train, the load on the preceding cross-girder being neglected.

In addition to the live loads before mentioned, the structure shall carry the following dead load, viz.:

At the panel points of the loaded chords:

First.—The weight of the floor (composed of *a*, the weight of the cross-ties used for the particular kind of floor adopted, and *b*, a weight of 140 pounds per foot lineal of track—covering the weight of rails, guard-rails, splices, spikes and bolts).

Second.—One-half the weight of the truss.

Third.—The weight of the iron floor system, if any.

Fourth.—The weight of the lateral system belonging to the loaded chord; and

Fifth.—One-half the weight of the sway bracing.

At the panel points of the unloaded chords:

First.—One-half the weight of the truss.

Second.—The weight of the lateral system belonging to the unloaded chord; and

Third.—One-half the weight of the sway bracing.

The span for calculation is to be taken from center to center of end pins, or from center to center of abutment plates or other supports; and the height from center to center of chord pins in truss bridges, or between centers of gravity of flange sections in plate girders, provided it does not exceed the distance out to out of angles, in which case the latter amount shall be considered the height.

The maximum and minimum stresses in compression and tension, as found for the before-mentioned loads, are to be used in determining the permissible working stress in each piece of the structure, according to the following formulas:

For pieces subject to one kind of stress only (all compression or all tension):—

$$a = u\left(1 + \frac{\text{minimum stress in member.}}{\text{maximum stress in member.}}\right) \qquad (1)$$

Fig. 1.

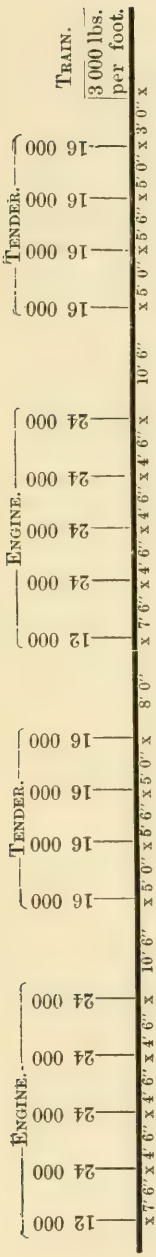


Fig. 2.

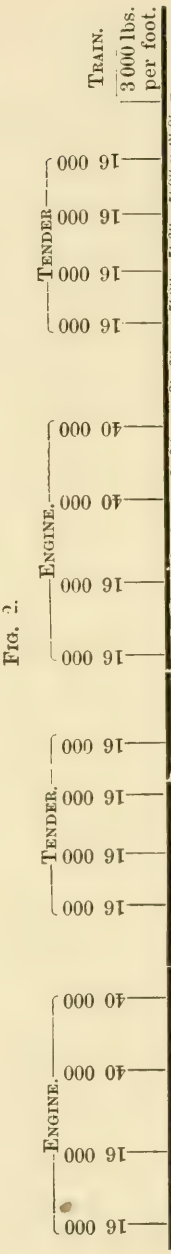
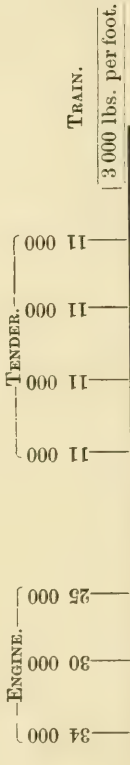


Fig. 3.



For pieces subject to stresses acting in opposite directions :—

$$a = u\left(1 - \frac{\text{maximum stress of lesser kind.}}{2 \text{ maximum stress of greater kind.}}\right) \quad (2)$$

In the above formulas :—

a = permissible stress per square inch, either tension or compression.

u = for double-rolled iron in tension (links or rods) 7 500 pounds per square inch.

u = for rolled iron in tension (plates or shapes) 7 000 pounds per square inch.

u = for rolled iron in compression 6 500 pounds per square inch.

The permissible stress a for members in compression is to be reduced, in proportion to the ratio of the length to the least radius of gyration of the section, by the following formulas :—

$$\text{For both ends fixed } b = \frac{a}{1 + \frac{l^2}{36\,000\ r^2}} \quad (3)$$

$$\text{For one end hinged } b = \frac{a}{1 + \frac{l^2}{24\,000\ r^2}} \quad (4)$$

$$\text{For both ends hinged } b = \frac{a}{1 + \frac{l^2}{18\,000\ r^2}} \quad (5)$$

Where a = permissible stress previously found.
b = allowable working stress per square inch.
l = length of piece in inches center to center of connections.
r = least radius of gyration of the section in inches.

Pieces used in compression which are continuous over points of support are to be considered as hinged at the ends, unless so firmly fixed in direction as to be incapable of bending in opposite directions on the opposite sides of the points of support :—

In all cases where possible, the lines of the neutral axes of all pieces meeting at a joint must be made to meet in the same point, and where pins are used to form connections, they must be placed as nearly as possible in the neutral axes of the sections.

When not so arranged, provision must be made for taking up the bending stresses produced.

When the floor system rests directly on the upper chords of deck bridges, the said chords shall be so proportioned that the algebraic sum of the stresses per square inch on the outer fibers [due,

First.—To the weight of that part of the floor system which is supported by the chord (considered as acting on a continuous beam of a span equal to the panel length).

Second.—To the direct thrust.

Third.—To three-fourths of the maximum bending produced by that portion of an engine of the heaviest class which is supported

by the chord on a span equal to the panel length (considered as a supported beam); and

Fourth. (In case the pin is not in the neutral axis of the chord.) —To the algebraic sum of the moments of all chord stress increments acting at centers of pins.]

shall not exceed at the panel point the working stress a, or shall not exceed at the center of the panel the working stress b.

All other members which are subject to direct stress in addition to bending moment are to be similarly calculated.

Built chords must be thoroughly spliced and the splices riveted in the field, not bolted.

The eyes on all tensile members shall have fifty per cent. excess of material at the pin when the diameter of the pin does not exceed the width of the bar, and one hundred per cent. excess when the diameter is twice the width of the bar or over. For intermediate sizes of pins the excess of eye may be made proportional to their diameter.

The diametrical bearing between pins and pin-holes (diameter of pin $\times$ thickness of bearing) shall not be less in area than

$$\tfrac{2}{3}\left(\frac{\text{maximum stress in member.}}{\text{compressive unit stress } a \text{ for that member.}}\right)$$

Eye plates must have a sufficient size and number of rivets to properly distribute the bearing stress from the pins to members of the truss.

Pins are to be so proportioned that the maximum stress per square inch on the outer fibers (calculated from the cumulative moments of the stresses acting on the pieces connected, and the moment of resistance of the pin directly) shall not exceed one and one-half times the maximum tensile stress a in the members connected.

All rods with screw ends must be upset, and if ordinary nuts are used they must be double. Rods provided with adjusting screws are each to have an amount of five tons added to the calculated stress to allow for initial stress.

Floor-beam hangers must have an additional section of twenty-five per cent. above that given by the before mentioned limiting stresses.

Rivets must be so spaced that they shall not be further apart in the direction of the stress than twelve times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of stress.

Rivets must be kept a sufficient distance from the sides and ends of pieces to avoid any danger of splitting out, and not placed closer than three diameters center to center.

Single lattice straps shall have a thickness of not less than one-fortieth ($\frac{1}{40}$), and double straps connected by a rivet at the intersection, not less than one-fiftieth ($\frac{1}{50}$) of the distance between the rivets connecting them to the compressed members; and their width shall be :

For 15-inch and 12-inch channels, or equivalent built section ($\frac{7}{8}$-inch rivets), $2\frac{1}{2}$ inches.

For 15-inch and 12-inch channels, or equivalent built section ($\frac{3}{4}$-inch rivets), $2\frac{1}{4}$ inches.

For 10-inch and 9-inch channels, or equivalent built section ($\frac{3}{4}$-inch rivets), $2\frac{1}{2}$ inches.

For 8-inch and 6-inch channels, or equivalent built section ($\frac{3}{4}$-inch rivets), 2 inches.

For 8-inch and 6-inch channels (extra light sections) and 5-inch channels ($\frac{5}{8}$-inch rivets), $1\frac{3}{4}$ inches.

The distance between connections of the strapping shall be such that the individual members composing the column considered with hinged ends and a length equal to the distance between these connections shall be stronger than the column as a whole; and in no case shall this distance exceed eight (8) times the least width of these members.

All segments of members in compression connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum stress borne by the segment, and the thickness of which shall not be less than one-fortieth ($\frac{1}{40}$) of the distance between the rivets connecting them to the compressed members.

The shear on net section of any member shall not exceed the compressive working stress a, and in case of rivets at least twenty per cent. extra section must be allowed.

No allowance shall be made for the web in calculating the flange sections of plate girders.

The stresses in solid rolled beams shall be calculated from the moment of inertia of the section.

Flanges of plate girders running over twelve inches in width shall have at least four lines of rivets.

The stress in the outer fibers of I beams, channels, etc., subject to bending moments, shall not exceed the tensile working stress a for rolled shape iron.

In all cases for compressed flanges of beams or girders (subject to transverse stress), the permissible working stress in such flanges shall be computed by Rankine's formula :

$$c = \frac{a}{1 + \frac{l^2}{5000\ w^2}} \qquad (6)$$

Where a = permissible stress previously found.
c = allowable working stress per square inch.
l = unsupported length in inches.
w = width in inches.

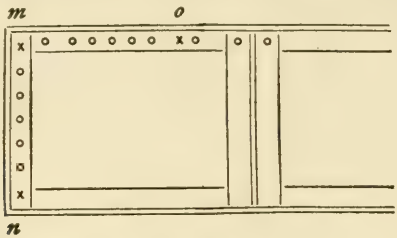
In no case shall a stress greater than that for a length equal to twelve times the width be used.

The rivets in plate girders shall be proportioned for shear as previously specified, and the rivets through the web and flange angles, and through the web and vertical stiffeners, at a splice, concentrated load or end of girder, must not have a bearing pressure per square inch against the web plate (on a diametrical section of the rivets) of more than twice the compressive stress a used in the upper flange of the girder; provided that they are not placed closer than twice their diameter from the edge of the web-plate to edge of hole. If placed closer than this the bearing pressure must be proportionally reduced.

In calculating the shearing stress and bearing stress on web rivets of plate girders, the whole of the shear acting on the side of the panel next the abutment is to be considered as being transferred into the flange angles in a distance equal to the depth of the girder.

The number of rivets in the flanges in the distance $m - o$ (equal $m - n$) shall be sufficient to transfer the shear at $m - n$ into the flange angles without exceeding the specified shearing stress or the bearing

stress on diametrical section of rivets, and the number of rivets in the distance $m - n$ shall follow the same rule.



Net sections must be used in all cases in calculating tension members, and in deducting rivet holes they must be taken one-eighth inch larger than the size of rivets.

In calculating the net section of angles in plate girders *all* the rows of rivets must be deducted, and in flange plates having rivets staggered, all rows must be deducted unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate.

When the thickness of the web plate is less than one-thirtieth of the unsupported distance between flange angles, heavy stiffeners shall be riveted on both sides of the web with a close bearing against the upper and lower flanges, and calculated as columns for the whole shear at the several points where they are placed. These stiffeners, in girders over three feet in depth, shall be placed at distances apart (center to center) generally not exceeding the depth of the full web-plate, with a maximum limit of five feet. In girders under three feet in depth they may be three feet apart, and in some special cases, where there is little or no shearing stress, at a greater distance.

In every case at least one upper flange plate on plate girders shall extend from end to end of the same to give lateral stiffness, and any additional plates used to make up the flange section shall be made of such length as to allow of at least two rows of rivets, of the regular pitch, being placed at each end of the plate beyond the theoretical point required.

Girders formed with web-plate and angles alone, having no upper flange plate proper, will not be allowed.

All flange plates, subject to either tension or compression, spliced in the length of girder, must be covered by an amount of extra material equal in section to the pieces spliced, with sufficient rivets on either side to transmit the stresses from the parts cut.

Flange angles must be spliced with angle-covers whenever cut within the length of the girder, or else the amount of material cut must be replaced by an equal amount of extra material in the flange plate.

No iron shall be used of less thickness than one-quarter inch, no piece used in compression shall have an unsupported width of more than thirty times its thickness, and no plate girder web shall be less than three-eighths of an inch in thickness.

Continuous girders will not be permitted, except in the case of upper chords carrying floor and in drawbridges.

Through bridges must have a clear head room of not less than eighteen feet six inches from base of rail.

Standard inside clear width for full through bridges, single track on a tangent, is thirteen feet. Width for bridges on curves and half through bridges where girders project up above the track, is subject to modification, depending upon the special case.

Truss bridges are to be cambered with a rise of not less than $\frac{1}{1200}$ of their length; the cross ties to be sized down over the track girders, so that the camber line of track may be a true circular arc.

Cross ties to be of white oak, having a width of ten inches and a minimum depth of seven inches, and spaced not over twenty inches between centers, with every fourth tie bolted down by three-quarter inch bolts, having round flat heads and two hexagon nuts each. When track is curved, the outer rail to be elevated as may be required.

In the case of deck bridges with wooden floor-beams, when the distance between centers of supports exceeds six feet, the floor-beams (ties) are to be proportionally increased.

Guard rails of white oak or long-leaved southern pine, six by eight inches, are to be placed ten inches in the clear outside of each track rail; to be notched one and one-half inches over the cross ties, secured to every fourth tie by a three-quarter inch bolt, having flat round head, two hexagon nuts and flat wrought washer, and to all other ties by three-quarter inch square wrought spikes. Splices to guard rails to be twelve inches long, placed between ties, with the joint horizontal, two three-quarter inch bolts with flat round heads, wrought washers and double hexagon nuts being used for each.

Lateral bracing shall be proportioned for a wind pressure, acting in either direction horizontally, of thirty pounds per square foot on the whole surface of all trusses and the floor, as seen in elevation, in addition to a train of ten feet average height, beginning two feet six inches above base of rail, moving across the bridge; except in the case of through bridges, where the surface of the truss covered by the train may be deducted.

Where the bridge is on a curve, the lateral bracing, in addition to wind stress, must be proportioned to resist a centrifugal force due to as many trains as there are tracks, moving at the rate of sixty feet per second.

The whole of the wind stress due to the train and floor, plus one-half the truss, is to be considered as acting on the lateral bracing of the loaded chord, and that due to one-half the truss only on the lateral bracing of the unloaded chord. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind stress from the upper lateral system to the end posts, and the end sway bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal stress from the loaded chord to the abutment, intermediate sway bracing being placed in each main panel and adapted to carry half the maximum stress-increment due to the wind on the train and to centrifugal force. In case of very heavy curves, some of the centrifugal force may be transferred to the lower lateral bracing.

In through plate-girders the portion of the train covered by the girder may be deducted from the wind surface, and only one girder and one train surface considered.

In all cases where the rods have adjustment, an addition to the above stresses of five tons must be made for initial tension.

Lateral rods in tension shall not be strained more than 15 000 pounds per square inch, and plate or shape iron not more than 12 000 pounds per square inch under the above conditions.

Lateral struts in compression shall not be strained more than 12 000

pounds per square inch (including the proper component of the initial tension allowed on the lateral rods at their extremities), and reduced in proportion to their length and least radius of gyration, as previously specified.

In case the maximum stresses in the chords of the bridge or flanges of floor-girders due to wind and centrifugal force (the chords and lateral bracing being considered as a truss lying on its side), added to the maximum stresses in the chords or flanges due to vertical loading, shall exceed the before mentioned limits of 15 000 and 12 000 pounds per square inch in tension and 12 000 pounds per square inch in compression, properly reduced, additions must be made to said chords or flanges until such limits are not exceeded.

Should the stresses in said chords be reversed in any possible case, proper provision must be made for such stress in an opposite direction.

No deduction shall be made from chord sections on account of material in lateral system, but the chords shall be made of the full section previously specified.

In every case the connections between the wind bracing and chords must be made of greater strength than the wind bracing itself, and so designed as to avoid as far as possible inducing bending moments in any members of the structure, and such connections must be capable of transferring the longitudinal components of the wind stresses into the main truss chords in a direct and satisfactory manner, or a separate chord be used for the lateral system.

The trusses must be secured from side motion on bearing plates, and must have ample bearing and roller support, the weight on the rollers not to exceed $750\sqrt{d}$ pounds per lineal inch, d being the diameter in inches. Girder bridges less than sixty-five feet opening will not require rollers. The bolster blocks must be joined to the truss. The bearing plates must be secured to the underlying support by bolts or dowels. Bearing plates shall not give a greater pressure on masonry than 300 pounds to the square inch, unless in specially authorized cases.

In the case of trestles or iron piers, they shall be proportioned for vertical load under the same limiting stresses given for trusses, and for wind stresses and centrifugal stresses, loading and bending combined, the stresses shall not exceed those given for lateral bracing.

In addition to the above, the structure shall be capable of resisting wind pressure on its exposed surface alone of fifty pounds per square foot without exceeding the limiting stresses for lateral bracing.

Tension at the foot of the windward column is to be avoided if possible, and in any case approved anchor bolts well secured in the masonry shall be used.

All strain sheets and plans must be submitted to the engineer of the company in charge of bridges for approval, and a complete set, including details, furnished to him without charge. All details to be subject to his approval, and access to be allowed him or his assistants to the contractors' working drawings and shop for examination of details.

Quality of Material.—All wrought-iron must be tough, fibrous, uniform in quality throughout, free from flaws, blisters and injurious cracks, and must have a workmanlike finish. It must be capable of sustaining an ultimate stress of 46 000 pounds per square inch on a full section of test-piece, with an elastic limit of 23 000 pounds per square inch.

All iron to be used in tension or subjected to transverse stress (except web-plates), must have a minimum stretch on a length of eight inches of fifteen per cent. measured after breaking.

All iron to be used in compression and for web-plates, of width not exceeding twenty-four inches, must have a minimum stretch of ten per cent. on a length of eight inches measured after breaking.

All iron for web-plates exceeding twenty-four inches in width, must havea minimum stretch of five per cent. measured in length of eight inches.

All iron to be used in the tensile members of open trusses, laterals, pins, bolts, etc., must be double-rolled after and directly from the muck bar (no scrap will be allowed) and must be capable of sustaining an ultimate stress of 50 000 pounds per square inch on a full section of test-piece, with an elastic limit of 25 000 pounds per square inch and a minimum stretch of twenty per cent. measured after breaking in length of eight inches.

When tested to the breaking, if so required by the engineer, the links and rods must part through the body and not through the head or pin-hole. Such tests must be at the expense of the contractor when the requirements of these specifications are not complied with.

All tension wrought-iron, if cut into testing strips one and one-half inches in width, must be capable of resisting, without signs of fracture, bending cold by blows of a hammer, until the end of the strips form a right angle with each other, the inner diameter of the curve of bending being not more than twice the thickness of the piece tested. The hammering must be only on the extremities of the specimens, and never where the flexion is taking place. The bending must stop when the first crack appears.

All the tension tests are to be made on a standard test piece of one and one-half inches in width, and from one-fourth to three-fourths inches in thickness, planed down on both edges equally, so as to reduce the width to one inch for length of eight inches. Whenever practicable, the two flat sides of the piece to be left as they come from the rolls. In all other cases both sides of the test pieces are to be planed off.

All plates, angles, etc., which are to be bent in the manufacture must, in addition to the above requirements, be capable of bending sharply to a right angle, at a working heat, without showing any signs of fracture.

All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending until the sides are in close contact, without showing fracture on the convex side of the curve.

Pins of four and a half inches diameter or less may be rolled iron, but those of greater diameter must be forged.

Workmanship.—All workmanship must be first class; all abutting surfaces, except flanges of plate-girders, must be planed or turned so as to insure even bearings, taking light cuts so as not to injure the end fibers of the piece, and must be protected by white-lead and tallow. Abutting members must be brought into close and forcible contact when fitted with splice-plates, and the rivet holes reamed in position before leaving the works, the plates being marked so as to go in the same position in erecting.

Generally the use of bolts instead of rivets will not be permitted, unless they are turned conical and the holes reamed to fit them.

Rollers must be turned and roller-beds planed.

Rivet holes must be carefully spaced and punched, and must in all cases be reamed to fit, where they do not come truly and accurately opposite, without the aid of drift pins. Rivets must completely fill the holes and have full heads, and be countersunk when so required, and machine driven wherever possible.

Compression members must be straight and free from kinks or buckles in the finished piece.

All pin-holes in pieces which are not adjustable for length must be accurately bored at right angles to the axis unless otherwise shown in the drawings, and no variation of more than one-sixty-fourth of an inch will be allowed in the length between centers of pin-holes. Eye-bars must be perfectly straight before boring; the holes must be in the center of the head, and on the center lines of the bar. Whenever links are to be packed more than one-eighth of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended position before being bored, suitable blocking pieces being used to keep them in proper position during the operation of boring. All pieces must be at equal temperatures when bored, and those belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. Pins must be carefully turned, perfectly finished and straight, and when driven in must have pilot nut to preserve threads. No variation of more than one-thirty-second of an inch will be allowed between diameter of pin and pin-hole. In the case of bolts, a variation of one-sixteenth of an inch will be allowed between diameter of bolt and hole. Thickening washers must be used whenever required to make the joints snug and tight.

All iron must receive one coat of raw linseed oil as soon as received at the works, and a coat of approved red oxide of iron before leaving the works. All inaccessible surfaces are to be painted with one heavy coat of red oxide of iron in pure linseed oil. All iron to be scraped clean from the scale before painting.

The whole of the construction to be first-class work, and in strict accordance with the drawings and these specifications. In the case of sub-contractors, the specifications are fully binding on them in every respect, and free access and information is to be given by them for thorough inspection of material and workmanship, and all required test-pieces, etc., properly shaped, are to be provided as may be requested, without charge. All shipments of material not properly inspected and passed are at the risk of the contractor.

In all cases figures are to be taken in preference to any measurements by scale.

No alterations are to be made unless authorized by the engineers.

DISCUSSION.

Discussion by Theodore Cooper, W. Howard White, G. Bouscaren, William H. Burr, S. W. Robinson, Geo. L. Vose, J. B. Davis, Geo. F. Swain, Mace Moulton, A. P. Boller, Mansfield Merriman, James G. Dagron, William Sellers, Edwin Thacher, George H. Pegram, C. C. Schneider, T. C. Clarke, and Joseph M. Wilson.

Theodore Cooper, M. Am. Soc. C. E.—The purpose of presenting this paper before the Society was, undoubtedly, to obtain the benefit of criticism from bridge experts in regard to the accompanying specifications. This paper is one on which there is a decided difference of opinion, and I hope there will be a full discussion upon the subject. My own belief and practice is totally at variance with many of the views herein advocated by the author.

For obtaining the unit strains to be used in proportioning the various members and details of a bridge, the author has introduced into his specifications the form of formulas advocated by Launhardt and Weyrauch as deduced by them from the consideration of Wöhler's experiments.

Of the value of these experiments of Wöhler there is no question. They are the most interesting set of experiments ever made upon the strength of materials, being made in a manner to tax the material in a like manner to that of actual use. The results are remarkable and deeply interesting. They are, however, too limited to satisfy our desires. Theorists have endeavored to fill the want, by deducing formulas from these limited experiments to cover all possible cases in practice. Such are the formulas of Launhardt and Weyrauch. They have been before the profession for some years, and have been pretty thoroughly discussed, notably between the French and German engineers for the past five years.

At the best, they simply represent the breaking strains; whereas, it is now generally accepted, that for permanently safe structures we must have regard solely to the permanent elastic condition of every member.

Wöhler's experiments show very conclusively that while good iron, strained as usual in our testing machines, may have a certain range of elasticity from a zero of strain, this range of elasticity only holds for other kinds of straining by measuring it from the initial condition of the test piece. For example, for material having a range of elasticity of 34 000 pounds per square inch, when measured from zero the range will only extend from — 17 000 to + 17 000 when the material is subject to alternating strains. This is not represented by the above-mentioned theoretical formulas. They cannot therefore be accepted in their full significance. Hence they should not be used as the author uses them. I also object to their use for other reasons.

First.—They leave in too indefinite a shape the definite instructions which should be put in general specifications for the use of those who are to bid upon any work; leaving too much room for varying interpretations by different designers, thus preventing a just and fair competition.

Second.—They tend to obliterate the judgment and responsibility of the engineer, and to place the same upon the contractor, where it does not belong.

The author in like manner treats the question of impact, and, as before, I object to the method. We do not know the effects of impact upon our structures and their individual members with sufficient definiteness to formulate any law, nor do I think we can define a required percentage for the different members in such a manner as to bring a number of different proposals to an equal and just comparison. The form and construction of any member has also a large influence upon its ability to resist impact. A built member formed by riveting a number of pieces together cannnot resist such action as well as one homogeneous body, nor will a short suspender as well as a long one.

In all such matters we must be guided rather by experience than theory; our judgment can be guided by the experience of others and also by due consideration of even imperfect theories such as Launhardt's formulas and the various allowances for impact. The results should then be clearly and definitely stated in our specifications.

My own specifications for bridges, which are at least in as extensive use as any other in the United States, are based upon this belief. The unit strain to be allowed upon each kind of member is definitely stated, all allowances being given due consideration before selecting the same.

Another merit of this method is, that the engineer can so assess the unit strains upon undesirable forms as to favor what he believes to be the best practice.

I believe I can claim to have drawn the first bridge specification (Erie Railroad Bridge Specifications, 1879) in which there was no recognition of the so-called factor of safety or "factor of ignorance." This led necessarily to the definite selection of the unit strains for each particular case. The results of my practice and observation since have satisfied me of the desirability of thus definitely defining all unit strains without possibility of misconstruction.

For identically similar reasons I object to the specifying of the unit force as so much per square foot of the exposed surface, and the determination of the lateral stability from such data. It is not correct, and is too indefinite. Lateral strains are due both to wind and the oscillations of the trains. With our usually accepted practice, it is fully as correct and far more definite to specify a fixed amount of lateral force per lineal foot of the loaded and unloaded chords, based upon the results of accumulated experience. I have endeavored to make clear in a general way my ideas as to the principles upon which specifications should be based.

There are many matters of detail in the specification submitted which I could not approve, but it will not be necessary to enter upon these, as they are clear to any one upon comparison with my own specifications. I regret that the author has not expressed his reasons for the following clause: "Girders formed with web plate and angles only, having no upper flange plate proper, will not be allowed." This is so contrary to the practice of every other designer, that it should have a reason. My own practice is never to use any flange plates if angles can be obtained heavy enough to give the required sectional area.

The author in his paper refers to an article of mine published some eight years ago, on the principles governing the thickness of the web pitch of rivets and stiffeners in plate girders. He is wrong in his conclusions therefrom, that I would "place stiffeners 2½ feet apart" in a girder 60 inches deep and 60 feet long.

While the principles are correct from which he deduces this result, I would never in my practice use it to any such extreme. I have seen girders of this kind, but they were made by those who do not understand the principles of designing a plate girder, in my opinion. The

web has a certain duty to perform in transferring the shearing strains, both vertical and horizontal. The horizontal shear between the web plate and the flange angles determines the number of rivets at each part of the girder. Practical considerations determine the number of rivets that can be placed within a definite limit of the angles (about 3-inch pitch for a single row of rivets and about 2½ in double rows). The bearing pressure on the rivets is usually limited to 12 000 pounds per square inch. With rivets then at 3-inch pitch the horizontal shear is limited to 4 000 pounds per square inch of web. The vertical shear is equal to the horizontal shear.

Using a shear of 4 000 pounds, our web plate would be about ⅜ thick, which would be abundant if there was not a liability for the web to crumple under this amount of compression. This may be overcome in one of three ways, or a combination of the three:

First.—Increase the thickness of the web to the ½-inch thickness, as given by the author.

Second.—Add stiffeners at intervals (my own practice is at intervals about equal to the depth of girder) until a point nearer the center of the girder is reached, where the original web of ⅜ is stiff enough to resist the shear at that point.

Third.—Reduce the depth of the girder (not desirable in the example given).

Fourth.—Use a thickness of web intermediate between the two extremes with a less amount of stiffeners.

For each case in practice the designer must select the method giving the most economical results, including cost of manufacturing. Frequent cases have come to my notice where the stiffeners added to the webs have contained enough material to have made the web throughout of the maximum thickness, and have not only saved the workmanship put upon the stiffeners, but placed the metal in the web where it would have added considerably to the strength of the girder (though we do not usually allow it to be estimated).

While Mr. Wilson's idea, that many girders act as Pratt trusses, is perhaps correct, I do not consider it good practice to design the girders upon this idea.

When we look back only a few years, and see how crude were the specifications upon which so many of our older bridges were built, it is a wonder the work was done as well as it was, and little wonder that

they are now proving their defects under the present increased loadings.

These specifications then consisted essentially of "bridge to be of such a length; rolling-load, 2 000 to 2 500 pounds per lineal foot; and a factor of safety of 5; workmanship, first-class."

It is too often the case now to see specifications which are simply compilations from other specifications (frequently good ones), but made in such a manner that they are contradictory and unintelligible, clauses being omitted, because not understood perhaps, which modify or complete other clauses which are selected. I have read Mr. Wilson's specifications, and I think my criticisms are based upon a correct understanding of them.

Mr. Wilson's experience has been a limited one in one sense. I do not say this to reflect upon Mr. Wilson, for we all recognize his ability and his standing in the profession. But as engineer of bridges for the Pennsylvania Railroad, his designs have been built by a limited number of builders, and those of the best class, and he has also been largely the designer of the works he has constructed. He has not had that contact with inferior builders necessary to discover how ingeniously they can take advantage of the weak points of a specification.

A specification upon one's own designs and confined to only the best class of manufacturers, need not be as definite or as strong as one for general bidding, or subject to the interpretation of other engineers.

An experience of a wide range has led me to provide for such cases, and thus to introduce clauses which are looked upon as unnecessary by those who have not had the same experience.

Many a contract has been decided in favor of a particular bidder who was sharp enough to take advantage of some clause badly expressed, or some requirement *expected*, but not definitely specified.

It is the engineer's duty to remove these questions of doubt, remove theories and formulas subject to misinterpretation, and state definitely and fully what he wants, so that the reputable and conscientious contractors have a fair and open field for competition against those who are less scrupulous or intelligent.

First-class builders like a strong, clear and full specification, intelligently interpreted.

W. Howard White, M. Am. Soc. C. E.—Mr. Wilson prohibits the allowance of web in calculating plate-girder flanges. I suppose this must be for convenience, or does he in constructing his theory of girder-web action abandon entirely the old method of calculation, as a beam with notches cut out of the sides?

Does Mr. Wilson extend his prohibition of girders without top plates over angles to floor beams whose top flanges are well held in line by stringers or floor plating over them, and if so, why?

Is he not a little rigorous in exacting flange-joint covers in all cases in compression? Such plates would give rather patchy looking work in long tower posts. Would not planed and well butted, and carefully broken joints do nearly as well constructively, and give neater work? I must take exception to his width between track-tie centers giving clear spaces of ten inches.

To my mind these spaces should be made as small as practicable. The limit should be such width as will allow of examination of the structure immediately under the track, such as rivets and laterals, without being slung below, and it should also not be so narrow as to risk holding coals dropped from fire-boxes in passing. I adhere to the 5 inches which I have previously advocated for clear space in bridge floors, and I have also seen no reason, with lapse of time, for abandoning my preference for inside guards where only one kind are used.

Mr. Wilson places the limit of pressure on masonry, 300 pounds per square inch, very low. This gives only 21½ tons per square foot, a weight which fairly gook brick-work can carry, and any good building stone will carry five to twenty-five times as much. The objection to such a low limit is the excessive size of bed-plates required, particularly in high iron-pier bearings.

Before seeing Mr. Wilson's paper I had intended making some observations on wind strains in continuation of the discussions in previous years on the papers by Messrs. Ashbel Welch, C. Shaler Smith, and F. Collingwood, Members Am. Soc. C. E., and as they had hardly seemed of sufficient value or importance to put forward as a separate paper, I am glad to hitch on to Mr. Wilson's more powerful train.

In considering wind strains there is a good deal of latitude as to how much wind pressure should be allowed, and as to the nature of the distribution of the same.

For instance, we in the United States nominally allow as a rule 30

pounds per square foot of exposed surface as a maximum, while the new Forth Bridge is being built on a 56-pound basis, and in Germany the practice is very various.

The first question which presents itself is, What is to be considered as exposed surface?

Probably, on inquiry of bridge engineers, nearly as many different statements of the manner of making this allowance would be got as there were answers. They would, however, differ mainly in such matters as allowance for smaller surfaces, such as laterals and sway braces, rails and adjoining guard timbers, which would not make a very material difference in the total wind pressure.

The allowance made at the Forth Bridge, as stated by Mr. Baker in his address before the British Association in 1884 (see *Engineer*, 29th August, 1884), is to be "fifty-six pounds per square foot, striking the whole or any part of the bridge at any angle with the horizon, and acting squarely or obliquely upon an area equivalent to twice the plane surface of the front girders, with the deduction of 50 per cent. in the case of tubes."

Mr. Baker used in his experiments a very ingenious arrangement for ascertaining the area of flat surface equivalent to the surface of his bridge members. This, as described in his address, consisted of a horizontal cross-bar suspended by a string at its middle point. A model of the portion of the bridge to be tested is placed at one end of the bar, and an adjustable flat surface at the other. On swinging the pendulum so arranged with the cross-bar at right angles to the plane of oscillation, if the surfaces balanced were not equivalent in resistance, one or the other would advance. The difficulty with this process seems to be to ascertain the center of pressure of the model with accuracy. How this was done was not stated. Possibly it might be determined exactly, but rather tediously, thus: First, assuming the center of pressure, clamp the model on the bar at a given distance from the string on a pin through the assumed center. Balance the model for pressure by a flat surface, with its center of pressure at same distance from the string as the pin. Then turn model partly round on pin and see if the result agrees with the previous one, adjusting accordingly until the model in any position with regard to the center, balances the flat surface. Mr. Baker's results are interesting, and in some respects surprising, inasmuch as he found that two plates corresponding to girders, connected by a floor plate at the bottom, gave only

90 per cent. of the resistance of a single plate. It is difficult to see how this can be. Mr. Baker's results on disks placed one behind the other, show the small resistance produced by our chord-bar arrangement as compared with European systems. He found that with two disks 1.5 diameters apart, the resistance to wind was one and a quarter times that of a single disk, and other disks could be introduced at will without increasing the resistance. In point of fact it would seem probable, from his results on the effect of connection plates, that intermediate plates would diminish rather than increase the pressure on two plates at a given distance apart, by preventing free access of the wind to the surface of pressure. Further particulars of the results of the experiments can be found in the above-mentioned paper.

A further question as to chord-bars is the effect of wind pressure on the outer one of a set in long panels. Where a single bar receives the pressure, and takes up the chord strain, it will yield laterally until the resistance to transverse strain, and its action as a catenary, together balance the rod pressure. The increase of tension in the stretched side of the bar is its only increase of fiber strain, since the aggregate tensile strain in the bar must remain the same—namely, that due to its action as a tension member of the truss—whether it be straight or bent by wind.

If, however, the outer bar be one of a set, as is usually the case, protecting the other bars from deflection by wind, but forced to extend itself in order to take up the transverse pressure of the wind, which would otherwise cause infinite tension in this bar, the question is much more complicated, and the strain produced more serious in amount—a very important factor, in fact, in determining what the maximum strain in the chord really is.

If one assumes that the outer chord-bar of a panel 25 feet long is deflected by the wind into contact with the next bar, say 1¼ inches; in order that the bar may be held in this position by catenary action, a certain longitudinal strain is required, producing a definite elongation. A rough calculation of the effect of a 30-pound per square foot wind will show that the length of the arc formed by the bar with the deflection assumed, has not increment enough over the length of the straight bar to produce this elongation. It follows, therefore, that the bar, with such a wind, must rest against the next one, which, in turn would be deflected; and the result of this action would be a slight deflection of the pins were it not that the same action is taking place in the next panel. The whole problem thus

becomes excessively complicated. A solution of it, thoroughly in accordance with the spirit of our bridge building—and, indeed, with the spirit of our mechanical art in general, which aims at the avoidance of complicated situations as to strains or mechanism—would be the insertion of stops between the chord-bars. These might be arranged to project downward from a horizontal head lying on the chord. Or they might consist of a series of plates, clamped together by a bolt at top and bottom, and imparting a desirable stiffness to the chord, though the solution would be objectionable in appearance.

Before going further into the details of wind-pressure effect, I will return once more to the amount of allowance to be made. The absolute amount assumed per cubic foot is, in any event, somewhat arbitrary.

It would not be worth while, if indeed possible, to build bridges strong enough to resist the hurricanes they are occasionally exposed to in the United States. Thirty pounds probably covers all tempests which are not really phenomenal, and it is well to build bridges strong enough to resist such a wind pressure when unloaded.

The question, then, is: What pressure ought they to resist when otherwise fully loaded? A loaded box car will stand upon the tracks with over 30 pounds per square foot of wind pressure, but a man cannot remain upon his feet in such a wind, even though his body is horizontal, and his legs at an angle of sixty degrees with the ground. About twenty-six pounds is the limit for keeping his feet. Now, though a train might be found on a bridge when such a pressure occurred, it is hardly possible that an engine-driver would take his train on to a bridge while such a wind is blowing. Occasional gusts of such force would be resisted somewhat by the inertia of the bridge itself, and might be allowed to produce an effect on the loaded chords up to the elastic limit. Something, however, should be allowed in chord strains for a surplus strain due to wind pressure in excess of load strains, though, strangely enough, it is often not done, the bridge being proportioned for wind pressure in the laterals without any allowance at all in the chords.* It will be evident from the above, that while indorsing thirty pounds per

* Mr. Wilson neatly covers this point by providing that "in case the maximum stresses in the chords of bridges or flanges of floor girders due to wind and centrifugal force (the chords and lateral bracing being considered as a truss lying on its side, added to the maximum stresses in the chords or flanges due to vertical loading) shall exceed the above-mentioned limits of 15 000 and 12 000 pounds per square inch in tension, and 12 000 pounds per square inch in compression properly reduced, additions must be made to such chords or flanges until such limits are not exceeded."

square foot as a basis for laterals, I would proportion the chords for a wind pressure in excess of the strain produced by the load of something considerably less than thirty pounds—say twenty pounds. Practically all wooden bridges in the United States, and a large part of the iron bridges of the United States and Europe, are built in such a way that the wind pressures in the top chord are for the most part carried down to the lower chords by transversely straining the posts, and thence, through the lower laterals, to the abutments. At the same time these bridges are not proportioned against wind on any such basis.

Where the bridge portal is not strong enough to take up the transverse pressure produced by wind in the top-chord system, it is obvious that there is no advantage in increasing the sizes of the top diagonals toward the ends, since there is nothing for them to bear against. The laterals (diagonals) in such cases are merely an assistance in holding the top chord in line, and they can, in the case of a deck bridge thoroughly sway-braced, be perfectly well omitted, if it is desirable to do so for any reason, such as an exigency of floor construction.

In truss bridges, without strong portals, the rocking motion of the wind causes an increased percentage of load on the lee chord. If P is the wind pressure per lineal foot of bridge; h the height of its point of application above the chord center; a the width between chord centers; and W the weight per lineal foot of the load on each truss, the increased percentage of load on the lee chord is represented by $\frac{Ph}{Wa}$. This increase of load is to be treated exactly as ordinary load, since it is an absolutely downward pressure at the post foot, and independent of the horizontal effect of wind pressure, the whole of which must in such cases be taken up by the bottom chord.

If any one has any doubt as to the double increase of strain on the lower chord arising in this way, a simple experiment will, I think, dispel such. Take a block of wood, of which the length and width are proportional to the height and width of a truss, while the thickness of the block stands for a panel length or any other unit, and stand this on a table. The friction on the table then takes the place of the lower chord lateral trussing. If we attach a weight by a thread to the base of the block, pulling it transversely to the bridge section, we have a representation of the part of the wind pressure which is borne directly by the lower chord. A weight similarly connected to the top of the

block, and pulled in the same horizontal direction, represents top-chord wind pressure. If the weights are made sufficient to just not slip the block on the table, it will be found that it is immaterial in the effect whether both threads are attached to the bottom, or one to the top and the other to the bottom. It is therefore evident that when the strings are in the latter position, representing a bridge without top laterals, or a strong portal for them to abut against, the whole wind pressure is resisted by the bottom laterals, as was to be expected. But in this position it is also evident that the lee edge of the block gets an increase of load, which was also to be expected; but in view of the tremendous increase of strain thus brought upon the lee lower chord, the experiment is not uninteresting.

When this transfer of top-chord wind pressure to the lower lateral system is effected by short braces in the upper corners of the cross-section, the increased load is be reckoned into the load on the top of posts, and an ugly transverse strain in the posts is caused, which must be duly provided for. For this reason the more recent practice of heavy portals carrying the entire top-chord strain, leaving the post free from transverse pressure, is decidedly the better one, and will be found so also in deck bridges in a majority of cases.

Sway braces, so called, are, however, highly desirable in deck bridges; and in through bridges where the height is sufficient to admit of an overhead panel with horizontal strut top and bottom, and diagonal ties; first, to reduce the free length of the posts; secondly, in double-track bridges to assist in distributing the load over both trusses; and thirdly, to serve the same purpose in single-track bridges in high winds, when the live load is unequally divided upon the two trusses, through the effect of the wind moment on the train.

In deck bridges, where both reduction of post length and distribution of load is needed, sway bracing must, of course, be double-decked.

It will be evident that where inclined portals are used, we save, in addition to other savings, the top-chord strain over the whole length of chord due to wind pressure of one panel, but we throw an additional chord strain into the lower chord through the downward and longitudinal thrust of the lee posts from top lateral action.

Too great care cannot be given to the avoidance of transverse strains from wind-bracing connections, against which Mr. Wilson provides in a general way.

The strains that roll up in these members towards the ends of a large bridge, applied at an intersection with a floor beam or cross-strut center a few inches only from the chord, gives rise to tremendous horizontal moments of transverse strains on members which are often but little calculated for receiving it in that direction.

A corollary of this action is the twisting action produced by it on the feet of posts, and the consequent increase of tension on inside chord-bars.

Much study may be profitably expended to pass the centers of lateral ties as nearly as may be through the intersections of floor-beams or struts with the centers of chords, and in the same plane with the latter.

In this matter of intersections on center lines there is a good deal of slackness in girder practice—witness the lattice girders on the New York elevated roads, where the diagonal centers generally intersect the neutral axis of the chords at least six inches apart, manifestly causing very serious transverse strains in the latter members.

An interesting subject for any one mathematically inclined, will be found in determining what strains come upon the upper lateral rods, and what increase of strain on the verticals of a covered Howe bridge through the tendency of the wind pressure to rock the web system over into the plane of the floor. It is evident that this tendency has to be resisted: first, by increased tension in the vertical rods in preventing the extension of length of braces by rocking over on their lee corners; and secondly, by a bending strain in the top lateral rods at the points where they pass into the chords. This, of course, only where (as is almost always the case, however) these bridges have no sway bracing worth speaking of.

In view of the increased strain thrown upon the lower chords of bridges of this class by their mode of construction—first, by the transfer of the whole wind strain to the lower chord; and second, by the larger proportion of the load thrown upon the lee truss by the rolling moment of the wind—it is only surprising that the chronic weakening almost always found in these lower chords is not greater than it is.

This would be largely diminished by the adoption of portals framed to take up the lateral strains from the top chord without passing them through the lower chord.

In conclusion, I would like to call attention to discrepancies fre-

quently found in proportioning roofs, where allowance is made for 30 pounds, or even more, of wind, when the walls of the structures on which the roofs are placed, such as detached shops, railroad stations, etc., would not stand anything like such a pressure.

G. Bouscaren, M. Am. Soc. C. E.—*First.*—The principal feature of Mr. Wilson's specifications is the general application of Launhardt's formula for the determination of the allowable stress in all classes of bridge members. As this is a radical departure from the time-honored rule of the "ultimate resistance" or "limit of elasticity" reduced by a "factor of safety," it is interesting to inquire as to the title of the new rule to such an important succession.

Launhardt's formulas are the outcrop of Wöhler's and Spangenberg's experiments, made principally on tension and bending. The value of u for compression is not yet determined by experiment. It is simply assumed that this value is the same as for tension.

No experiments have been made for alternate tension and compression, except where the opposite stresses are equal. The formula for alternate stresses of opposite signs is deduced entirely by a process of reasoning from the results obtained in this particular case. The influences of rapidity of repetition, rapidity of increase of stress, and duration of individual stress have not been investigated.

The formulas do not take into account the influence of shapes and sizes of individual members. We all know, for instance, that the same working stress should not be allowed on a short suspension link as on a long tie rod; nor on a large tension member as on a rod of small diameter. Presumably repetition of stress must also give very different results for hard than for soft ductile metal, whether iron or steel.

In view of these facts, it would seem that the general application of Launhardt's formulas to the variety—in quality, shape, sizes and conditions—of bridge members is premature, and that adherence to the old rule, for the present at least, is the safer plan.

All that we can legitimately deduce now from the experiments of the German engineers, is that a strain within one-half of the elastic limit is allowable for any number of repetitions of the same kind; this is, in fact, nothing more than a confirmation of the old rule, with a factor of safety of one-fourth, inasmuch as the elastic limit is very nearly one-half of the ultimate resistance. This factor is generally adopted for all

parts of structures subject only to static loads. It is further reduced to one-fifth for structures, such as bridges, exposed to vibrations and contingencies beyond the reach of exact analysis, where the action of the elements, imperfect adjustment, and the secondary strains due to the deformation of the structure under load, all come in for their quota as disturbing elements.

Further than this, the old rule has the sanction of experience. This is certainly an important point in its favor.

I do not know of any instance where a bridge properly designed and proportioned, with a factor of safety of one-fifth, has failed, by reason of weakness, under the load it was intended to carry. In the present state of our knowledge on the subject, the general application of Launhardt's formula is in the nature of an experiment.

It is true that all advances in engineering are the fruits of experiments. But where safety is the prime consideration, it would seem that the most suitable place for experiments is the shop, and that the teachings thereof should only be carried into practice when shorn of their speculative element.

In the particular case of members subject to alternation of tensile and compressive stress, the ordinary factor of safety rule will not apply, as account must be taken of the special weakening effect due to the alternation of stresses of opposite signs. The usual practice of assuming for the working stress on such members the sum of the two maximums, leads in most cases to results evidently too large. Experiments to show the exact measure of the fatigue of the metal in such cases are very desirable; in the absence thereof, I have lately adopted the following rule, inspired by Launhardt's formula, for the determination of the allowable stress per square inch in both directions:

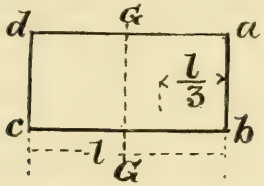
$$\text{For tension, } 10\,000\left(1-\tfrac{1}{2}\frac{s}{S}\right)$$

$$\text{For compression, } R\left(1-\tfrac{1}{2}\frac{s}{S}\right)$$

where s and S are respectively the smallest and largest of the two maximums, regardless of signs, and R the allowable stress for compression, calculated by Rankine's formula with a factor of $\frac{1}{5}$.

Second.—The live load adopted by Mr. Wilson is not likely to be exceeded in the near future, although some roads are, I believe, using consolidation and passenger engines with respectively 26 000 and 45 000 pounds on each pair of drivers.

Third.—The rule requiring the concordance of the line of stress with the line of center of gravity for each member is a very important one. The extent to which members may be overstrained by the unequal distribution of strain, due to the non-observance of this rule in designing details and connections, will be realized, if we remember, for instance, that in the case of a prismatic member of rectangular cross-section of width b and area Ω, the stress per square inch on the edge $a\,b$ is $2\,\frac{S}{\Omega}$, and on the edge $d\,c$, 0 (S being the total strain), when the center of stress is at $\frac{1}{3}$ of the width from $a\,b$; and when the center of stress falls on $a\,b$, then the stress per square inch is



on $a\,b$, $\quad 4\,\frac{S}{\Omega}$

on $d\,c$, $\quad 2\,\frac{S}{\Omega}$

Fourth.—With regard to the painting of bridges, there seems to exist some difference of opinion among engineers as to choice between oxide of iron and red lead. I have used both extensively, and give the preference to red lead.

On the Cincinnati Southern Railway, structures painted with three coats of the best quality of iron-clad paint had to be repainted the fourth year; structures painted with red lead in the sixth year. The latter were then in better condition than the former.

The red lead adheres better to the iron, and fails principally by wear and a gradual transformation of the red lead into carbonate, whilst the iron-clad fails by blistering and scaling, which requires the removal of the old material by scraping before a fresh coat can be safely applied. When the cost of this scraping, as well as the shorter life of the iron paint, are taken into consideration, the difference of cost of maintenance with the iron and lead paint is very much reduced, and there remains, in favor of the lead, a greater sense of security, arising from the fact that less damage will occur in case of neglect.

W. H. Burr, M. Am. Soc. C. E.—The fundamental principles of bridge design are very generally recognized at the present time to rest on a philosophical basis of principles, according to which the stresses

developed by exterior loading may in most cases be analyzed with sufficient accuracy for the production of a structure whose limit of resistance is essentially determinate. These excellent specifications demonstrate the feasibility of attaining this result, and show how thoroughly rational structural design has become, even in the two or three points in which it is open to criticism on purely rational grounds; for these very features are in the main avowedly theoretical. It is a matter of surprise, however, that a series of such excellent and minute directions for design should be preceded by an arbitrary approximation regarding the moving load, for it will be observed that after having placed the moving load in such a position that the locomotive drivers are over a cross-girder, the load on the latter shall be considered the head of the train, all preceding load being omitted. The reason for taking some other load than the actual, does not appear.

The methods for computing the absolutely greatest stresses for any system of wheel concentrations are so obvious and so simple, that it is difficult to imagine why they should be displaced by an approximation which may not always result in a safe error.

The recognition of the fatigue of metals, as shown by the tests of Weyrauch and Spangenberg, is certainly justified; but it should be borne in mind that they experimented on prepared test pieces, and not on finished bridge members. This is markedly true in the fatigue of pure compression, although it is probably much less than that of pure tension; in fact it is not an exaggeration to state that almost nothing is yet known regarding the fatigue of pure compression. It would seem, therefore, a little hasty to frame a formula for full-sized compression members from results on tensile specimens. In reality, it is probably most rational (as it is convenient) to fix the minimum working limit for the first counter and increase uniformly to the maximum value for the main tension member nearest the end of the truss, and proceed in the same manner for the compression web members.

The matter of web stiffeners for plate girders is one in which the author has widely missed the rational basis which he has certainly sought. The action of shearing stresses in web plates has been thoroughly understood since the establishment of the mathematical theory of elasticity in solid bodies, at a period so long previous to Professor Rankine's day that he acknowledges his indebtedness to Mr. Lamé. It would be interesting to learn the details of the experiment by which he

concludes "that the web resists by tension and not by compression." If the web of the paper model was made flexible by slitting it at 45 degrees (or any other angle) to the neutral line, it was obviously rendered incapable of resisting by compression, and the experiment would show nothing regarding the point under investigation. It is perfectly well known, and as firmly established as any mathematical truth can be, that with pure transverse shear, as at the neutral surface, equal intensities of tension and compression exist at 45 degrees to it and 90 degrees to each other. The corrugations of a buckled web plate verify these rational conclusions. It is an actual and absolute impossibility, therefore, that a web plate should resist by tension alone.

The tension and compression are inseparable, and must act together. In this fact lies the stiffness of such thin webs as mentioned by Mr. Wilson. The tensile stresses are at right angles to the assumed elementary columns (*i. e.*, to the compressive stresses), and hold them in place at every point of their entire length by a very considerable force, which must be overcome before buckling takes place.

Web plates, therefore, are far stiffer than would appear from the clumsy hypothesis of isolated elementary columns fixed at their ends only. Nevertheless there is a limit beyond which stiffeners must be used in order to retain the web in its own plane, although that limit is analytically indeterminate. But it is practically impossible that the strains or deformations in the stiffeners, rivets and web plate should adjust themselves with such nicety as is necessary to enable a stiffener to take stress as a strut, and it is a matter of great doubt whether such action would take place if such an adjustment were possible. If heavy concentrated loads are on the flanges immediately over the tops of closely-fitted stiffeners, the latter obviously may act as struts, but after the web has once taken the shear, it requires a very extraordinary condition of things to render a stiffener-strut possible. If the web plate can be retained in its plane, its resistance becomes equal to the pure shearing capacity of its tranverse section, and the office of the stiffener is no less important than to produce that result.

Just the frequency with which these stiffeners must be used, and their necessary sections, are yet matters of experience only. They belong to the many still open questions which may be qualitatively, but not quantitatively, answered. The ordinary and conventional rule used by Mr. Wilson for the spacing of the stiffeners is probably as

good as any, after they become necessary, but his sections are quite excessive.

The work to which these specifications, as a whole, lead, is certainly of a most substantial and satisfactory character.

S. W. Robinson, M. Am. Soc. C. E.—In a paper presented by the writer to this Society, on Vibration of Bridges, results of observations with a bridge indicator were given, in which the superadded strain, due to vibrations caused by the non-balanced locomotive drivers, was 28 per cent. of the maximum strain caused by the passing train when statically considered. Likewise an observed superadded strain, due to vibrations caused by the body of the train, was given, which was 50 per cent. of the strain due to the corresponding part of the train statically considered. Also, it was shown that certain diagrams taken gave statical strains for the train itself in excess of like strain due to the locomotive for the same trains.* Also, the observed fact that in practice train loads may exceed engine loads† was mentioned as a reason why the 50 per cent. should be provided for instead of the 28 per cent. in designing bridges where conditions favor vibrations from the train.

The bridge from which these particular per cents. were obtained was a through iron Pratt truss of 141-foot span, and 9 panels of 15⅔ feet each.

The above percentages are due to cumulative action or to a series of conspiring impulses, and therefore of much greater intensity than if produced by a single impulse of the same kind. Also, it should be noted that the percentages stated are not hypothetical ones, but are solid facts of observation read off from the direct and positive autographic records made by the bridge of its own vibratory movements occasioned by the passage of trains. Most of the 13 bridges from which autographic records were obtained, gave results of like kind, though not all of equal degree.

Some foreshadowing of these results was given in the report for 1881 of H. Sabine, Commissioner of Railroads for Ohio; also in Van Nostrand's *Engineering Magazine* and Science Series, and in numerous periodicals

* This statement of strains is made on the supposition that the greater strains are accompanied by the greater deflection.

† By "engine load" is meant the loading upon the bridge, consisting of engine and portion of following train.

by way of republication; but it may be presumed that some bridge specifications framed subsequent to those publications failed to give full credence to the effects whose existence was thus pointed out, for the reason that those effects were not sufficiently determined in degree to warrant it. But as results are now determined beyond question, and of by far too high a degree to ignore, this opportunity of calling attention to them is taken advantage of; but it is with the understanding that the specific results above mentioned were not made public until after the bridge specifications now being discussed were proposed, both being subject matter presented at the Annual Convention of 1885.

No provision is found in the specifications for the kind of dynamic effect above pointed out, except it be in a general way, as with all bridge specifications, by such excessive sections, or by low unit stresses, as is found in some unaccountable way to be necessary. For example: Why should a unit stress of 7 500 pounds be necessary for a member composed of first-class wrought-iron, when the experiments of Wöhler and Spangenburg show that such iron can stand for an indefinite number of repetitions a unit strain of about 30 000 pounds, unless it is to meet some practical demand the foundation for which is hitherto unaccounted for? If good iron will stand unit strains varied from 0 to 30 000 pounds repeated millions of times and yet be sound, where all strain is accounted for; then in bridges, if all strains are accounted for, why cannot the iron there work up to the same limits, or at least to two-thirds of it, something being allowed to make safety doubly sure? But a 20 000-pound unit stress is an unheard of permissible stress for a bridge in recognized good engineering practice.

But notwithstanding these extravagantly low unit stresses in present practice, iron bridges do sometimes go down, while the cause is passed as mysterious and unaccountable. How can it be otherwise than a stigma of disgrace upon the engineering profession that this hundredfold extravagance in use of iron exists to this day. Can it be otherwise than conceded by any rational mind that if all strains are accounted for, the working stresses of iron members may be carried up to at least two-thirds that which it is well known the iron will stand for repetitions well nigh infinite in number?

But, again, where unaccounted for strains exist, cannot they exist to greater intensity in some individual cases than in others without our knowledge, necessitating for a practical solution of the problem, the

adoption of less and less unit stresses, until such a gross allowance comes to be blindly made as to cover up all ignorance of strains. As long as one bridge in a thousand is subject to a 50 or 100 per cent. unaccountable strain, all must be so designed as to provide against that strain, and yet the unfortunate bridge whose unknown strain may reach a limit higher than tentative practice has made provision for, must go down; instances of which are well known. When an iron bridge goes down, the allowed working stress will go down, and so practice has continued until the present extravagant limit is reached of one-fourth that for infinite endurance according to definite experimental tests.

But fortunately a part of this hitherto undetermined strain has been made known through the investigation on bridge vibration, thanks to the persistence of Commissioner Sabine, of Ohio, for pushing the matter to the definite experimental results, already stated above, of 28 and 50 per cent. for the head of a train and body of a train respectively. Accounting for this latter percentage in bridges of 150-foot span, the allowed working stress may be raised about 30 per cent. for the same dead load, but in designing the bridge for a given live load, this percentage of increased unit stress will lighten the bridge and give occasion for a still further reduction of weight and corresponding decrease of strain upon the members. Suppose A to be the total possible strain likely to occur, and that the bridge will just be able to meet it on the basis of a 30 000-pound unit stress, then, with an allowed unit stress of 10 000 pounds, only one-third of the possible strain is accounted for and two-thirds not. But if 30 per cent. of the heretofore known strain be made known and added, it will be $\frac{1}{3} A + \frac{1}{3} A \cdot 30 = .43\, A$, or 43 per cent. of the possible strain is then known and can be accounted for, and accordingly the working stress can be carried up to 43 per cent. of the 30 000 pounds = 13 000 pounds, the additional 3 000 pounds being 30 per cent. of the 10 000 pounds, as above stated. Hence if all the now known strains be provided for, the allowed unit stress may be raised about 30 per cent. above values heretofore used with equal safety. For simplicity of illustration, these percentages have been figured on the basis of the formerly accepted unit stress, 10 000 pounds. Under the refinements of Wöhler's laws the figures would be slightly different.

But it may be said that we gain nothing by raising the stress unit along with the strain unit. If no effort is made to avoid the useless and unnecessary 30 per cent. strain due to cumulative vibration it is true,

but by so designing the bridge as to relieve it of this as much as possible, the increased unit strength may be employed with usual safety, and with a 30 per cent. saving in iron for main trusses.

To avoid the above mentioned 30 per cent. strain, cumulative vibration must be stopped, and this important step only requires that the panels of the bridge be in discordant relation to the half-car lengths, as pointed out in the paper on Vibration of Bridges. As no cumulative vibration was discovered for passenger trains, aside from the engine, we have only to consider the relation of panel lengths with freight cars. As the latter cars range in length between about 29 and 33 feet, then bridge panels should not be very near 15 or 16 feet, nor 7 or 8 feet. Probably 9 to 13 and 17 are the best panel lengths, as comparatively few coincidences occur with lengths between 9 and 13 or 17 and 20.

In deciding upon the panel lengths of any bridge, the prevailing length of freight cars, coupling to coupling, should be known. Then the half-car lengths may be compared with proposed panel lengths thus, for lengths continued in succession—

Half cars.....	0, 15, 30, 45, 60, 75, 90, 105, 120, 135.
Panels.....	0, 13, 26, 39, 52, 65, 78, 91, 104, 117.
"	0, 12, 24, 36, 48, 60, 72, 84, 96, 108.
"	0, 9, 18, 27, 36, 45, 54, 63, 72, 81.

Here we find that for the 13-foot panels the first coincidence after 0 is at about 100 feet from that point, which admits of but one or two coincidences on the bridge. In the 12-foot panel lengths we find coincidences at every fourth panel, and in the 9-foot panels at every third panel; the 13-foot panel being here preferred, and for which it is believed that cumulative vibration of considerable magnitude cannot occur.

If by making panel lengths 13 instead of 15 feet a 30 per cent. strain can be avoided, no one can reasonably deny the advisability of the 13-foot panel. This preference becomes all the more imperative when it is remembered that the cumulative vibration may go far beyond that as yet observed, and reach 100 per cent., or even more, as its possibilities are only limited by length of train, as pointed out in the original paper.

Indeed, beyond the now known 43 per cent., the entire balance of the rarely occurring, unaccounted for 57 per cent. strain above must be due to the same cause. In searching for the cause can we find any other than this one of cumulative vibration, and if we find no other, are we

not warranted in accepting this one as sufficient to account for that balance, when a previously unknown 30 per cent. portion has been actually brought within the range of positive fact from a few weeks observing, and when the possibilities of the same cause reach even beyond the whole balance of 57 per cent.?

In view of the above facts and figures relative to panel length, the necessity of a clause controlling panel length is strongly recommended for both economy and safety, in the present as well as all other railroad-bridge specifications.

By proper panel length and consequent avoidance of the strain caused by cumulative action of the body of the train, there will still be left the cumulative dynamic effect due to the unbalanced locomotive drivers of the above named 28 per cent. But fortunately in this case the possible impulses are determinate in number from the bridge itself, and much more limited than in the former case; and it will be much safer to assume a working limit for this strain than for the former, if indeed it be necessary to do so instead of adopting the advisable alternative of exactly counterbalancing the locomotive mechanism.

In deciding upon a safe limit, it must certainly be taken in excess of that observed within the few weeks observing, even for 150-foot spans, and a little consideration will show that it will vary with the span according to a law the precise expression of which will be considerably complex, dynamics making it about as the square of the span, while practical conditions would probably reduce it to about the first power of spans or a little greater. If it be granted that the possible effect for a 150-foot span is 50 per cent. greater than that observed (and I believe this to be none too large an allowance), the 28 will be raised at once to 42 per cent. of static strain due to live load for 150-foot spans, and for 300-foot spans 84 per cent., or about 28 per cent. for each 100 feet of span. This amounts to determining the live load L, where non-balanced drivers are employed, for use in calculating the bridge strains from the formula.

$$L = L_1 (1 + .0028\ S) \qquad (1)$$

where L_1 is the usual live load per foot run, and S the span of bridge. For a span of about 360 feet, this formula doubles the unit live load.

As to allowing for this strain due to cumulative vibration caused by the engine, and also that due to body of train, both in a given case, it seems not to be necessary, from the fact that they do not occur simultaneously, and the diagrams from the bridge indicator show that the vi-

brations caused by the engine hardly extend over into those due to the train. And as the latter, though largest from actual observation, are subdued by a judicious selection of panel length, the former strains only remain to be necessarily provided for in selecting the maximum load for calculation. If these strains be regarded as covering the whole ground of the hitherto unaccounted for strains, then the safe working unit stress may be carried up from the former 10 000 to say 20 000 pounds for the live load, equation (1), and with no hazard for safety.

From these facts and figures, it is believed that the present specifications, as well as all others, should embrace the above formula for determining the live load, as well as the clause for controlling panel length, and that the permissible working unit stress should be carried up accordingly.

As to eliminating the difference $L - L_1$ above in strain calculations, the civil engineer is at the mercy of the mechanical engineer, with little hope of relief, from the fact that at present there seems to be but little disposition on the part of motive-power superintendents to displace the present defective unbalanced mechanism by that which is balanced. But there can nevertheless be no question, that as long as the present objectionable mechanism is in use, the cost of bridge trusses must in consequence be increased by not less than about 18 per cent. for 100 feet spans; 30 per cent. for 200 feet spans; and 36 per cent. for 300 feet spans. In the meantime may all bridge specifications prepare for the advent of balanced locomotive machinery, by so fixing the panel length as to destroy the greatest of the two effects of cumulative vibration.

Relative to an attempt to provide for cumulative vibration strains along with the Launhardt and Weyrauch formulas, a little consideration will show that the formulas will require doctoring in the opposite sense to that practiced by Prof. Wm. Cain for including the impact as due to single impulses of various sorts; whereas, for the so-called impact, the greatest allowance is made for short spans, and vanishes at spans of about 100 to 200 feet. We have found that the cumulative vibration strains due to the engine's non-balance increase with the span; and by casting over the figures, it is seen that up to spans of about 150 feet, the total superadded strains to be provided for as due to both impact and vibration, amount to a nearly constant fraction of 50 per cent. of the strain due to the weight of the train, and more for longer

spans; the cumulative vibration due to the train being here and hereafter in this discussion regarded as avoidable. Up to about 150-foot spans then, the Launhardt formula should be applied to railroad bridges without modification as regards impact, while the provisional percentage is added to the load; and for longer spans the greater percentage. This would restore the coefficient $\frac{1}{2}$, which has been set aside by Prof. Cain, allowing that the $\frac{1}{2}$ generally used in the formula for iron is the best selection. But doubts may be expressed as to this value of the fraction, from the fact that it is much lower than that obtained by making use of all the existing data.

The value $\frac{1}{2}$ appears to have been adopted by Weyrauch with but little discussion, using a tensile strength of 46 800 pounds per square inch, a tenacity which would not be permitted for important tension members. Again, the fraction $\frac{5}{6}$ is offered by Spangenberg for this formula and obtained by Weyrauch, using a tensile strength of 57 160 pounds per square inch; a tenacity that is rather high, but indicative of better quality of iron than the former value. Why is the fraction $\frac{1}{2}$ adopted into the formula by Weyrauch for use by engineers, instead of the $\frac{5}{6}$, unless from timidity for departure from usual previous practice?

If the fraction $\frac{5}{6}$ be regarded as intrinsically better than the $\frac{1}{2}$, then the value 1 adopted into the formula for the Pennsylvania Railroad specifications is not very far from the $\frac{5}{6}$ found for good iron, leaving only $\frac{1}{6}$ instead of the supposed $\frac{1}{2}$ to account for the impact. In view of the value $\frac{5}{6}$, it may be questioned whether a higher value than 1, as $1\frac{1}{3}$, should not be taken in providing for impact.

From the vacillation of Weyrauch and others between the fractions $\frac{1}{2}$ and $\frac{5}{6}$, it seems desirable that some unquestioned value might be fixed. In searching for existing data to base a value upon, very little apparently can be found, and yet that, as meager as it seems, cost 12 years of experimental work by Wöhler, and was confirmed by Spangenberg. Hence the existing data, though scarce, is precious; and it seems hardly reasonable to adopt a constant almost by guess, to stand as the consideration for so vast an expense.

All the results accessible to me are:

	Max. Stress.	Min. Stress.
I.—For Iron—pounds per square inch,	$s = 16\,640$	$s = -16\,640$
	$u = 31\,200$	0
	$a = 41\,600$	$c = 24\,960$
	$t = (52\,000)$	$t = (52\,000)$
II. " " "	$u = 31\,300$	0
	$t = 57\,000$	$t = 57\,000$
III. " " "	$u = 31\,300$	0
	$t = 47\,000$	$t = 47\,000$
IV.—For Steel—pounds per square inch,	$s = 29\,100$	$s = -29\,100$
	$u = 49\,200$	0
	$a = 83\,200$	$c = 36\,400$
V. " " "	$u = 52\,000$	0
	$a = 72\,800$	$c = 26\,000$
	$a = 83\,200$	$c = 41\,600$
	$a = 93\,600$	$c = 62\,400$
	$t = 114\,400$	$t = 114\,400$
VI. " " "	$s = 22\,880$	$s = -22\,880$
	$u = 39\,520$	0

The values in *I* for iron are found in "Fatigue of Metals," by Spangenberg, except the figures in brackets, which have been supplied with the belief that they are about right for t, for the iron in question. The values *II* and *III* are found in Weyrauch, the former being that from which the value $\frac{5}{6}$ was obtained, and the latter the value $\frac{1}{2}$, by Weyrauch, for the Launhardt formula.

The values for steel are found in Spangenberg.

All these data are plotted on Plate XXXVIII A in 2 and 1 respectively, both in 3 (the square dot in 2 between s and u belongs to 3). Curves are carefully drawn through among the points, regardless of formulas, with the object of finding the best obtainable from the data. In 3, to which 1 and 2 are transferred, we see no very wide difference in general character of curve, both appearing to be parabolas. In plotting the curves, the maximum and minimum stresses are laid off from the zero line as indicated, each series of values being laid off in the order—circle, triangle and square for iron, and square, circle and triangle for steel; the "*I*" through the dots for iron, serving to distinguish iron from steel in 3, which shows the plottings for both iron and steel.

In plotting these curves, the line between c and d is taken at 45 de-

grees for convenience in plotting; c then being the distance from the foot of u, as well as the corresponding height to the 45-degree line.

For the steel, all the data are reduced proportionally to bring them about to the values for iron in order that they may be thus plotted in comparison.

At first it was proposed to adopt one curve for both iron and steel, but the absence of material advantages in this, and the prevailing difference of the curves, determined otherwise.

In examining these plottings and their curves, it would hardly seem necessary for better agreement of curves with points, that the portion between s and u differ from the rest; that is, that these parts follow different curves and laws; and, much less, be discontinuous at u. The plotted points for steel appear a little within the curves drawn at the point u, and it would seem that the curve might be straighter to advantage. But instead of a straighter curve, the present one has been insisted upon, for the reason that it is believed that the curve must not be discontinuous at s with that going to make the compression part of the diagram, and this requires that the tangent to the curve at s be parallel to the 45-degree line. Hence, the full line curves are believed to be the best that can be drawn in without introducing discontinuity at any point.

To show how these curves compare with those of the Launhardt and Weyrauch formulas, points have been determined by calculation from those formulas with their adopted fractions as found in Weyrauch, the points plotted and the curves drawn in with dotted lines for steel and iron at 1 and 2. These curves appear to be sadly wanting in both continuity and agreement with Wöhler's observed data.

In the Pennsylvania Railroad specifications the value of $\frac{u-s}{u} = \frac{1}{2}$, is the same as for the dotted curve between s and u for the curve 2, Plate XXXVIII A; but the adopted value of $\frac{t-u}{u} = 1$ in the same specifications, for the curve between u and t, is such as to place the curve further away in the direction of stress than the dotted line. This causes an excessive convexity from u to s; makes $s = \frac{u}{2}$; swells intermediate values between u and s out of all reason; and introduces a decidedly questionable intersection of the curves at u, or discontinuity.

From these considerations it would seem that the formulas are little better than guesses at the real requirements in the matter, and hence the

full line curves in Plate XXXVIII A have been drawn in as founded, as fully as possible, upon the experimental results of Wöhler.

When the curves were drawn, that for iron was found to be appreciably close to a parabola whose axis is coincident with the zero line of stress, and vertex at a distance at the left of u equal $\frac{3}{2}\,s$, while that for steel agrees well with a parabola whose axis is below the zero line. But when diagrams are used for determination of working stresses, there will be no need of formulas.

In the upper part of Plate XXXVIII A, the curve 2 for iron has been laid out along with all the quantities involved in the problem of determining working stresses, so as to indicate to the eye their true relation.

Plate XXXVIII B, for that part of the curve from t to u, is laid out according to the formula of Launhardt, where $\frac{t-u}{u} = 1$, as adopted in the Pennsylvania Railroad specifications. This curve (a parabola) is tangent to a line at 45 degrees at the top end of u, and from that point this tangent is used in this diagram from u to s, instead of the formula of Weyrauch for compression and tension by reason of the objectionable convexity given by that formula, such convexity not only causing discontinuity at u, but giving, as is believed, excessive values to a, between u and s. This diagram is believed preferable to the formulas found in the specifications from which to make out the working stresses when the maximum and minimum strains are determined.

To construct a diagram from which a may be read off, it is necessary that a be given in terms of the ratio of c to a, or of minimum B to maximum B, because practical values of minimum B and maximum B for a bridge member may be found in values a hundred-fold greater than the respective values of c and a, and yet have the same ratio. A good diagram for this purpose is of the form given in Mr. J. M. Wilson's statements respecting the Pennsylvania Railroad specifications.

A diagram of this kind is given in Plate XXXIX, suitable for use in office practice, except that it will be advisable to draw it to a larger scale. This diagram is drawn from values read off from the upper part of Plate XXXVIII A, and hence Plate XXXIX may be regarded as realizing that diagram which works with nearest approach to Wöhler's laws and experimental values. The working stresses a, on the left, are figured with a factor of safety given by dividing the value of u from Plate XXXVIII A, viz.: 31 200 by the corresponding value found at the intersection of the

curves in Plate XXXIX, viz.: 11 400, giving $\frac{31\ 200}{11\ 400} = 2\frac{3}{4}$ nearly. But where all strains are accounted for, as proposed in this discussion to be done, by taking the panel length such as to destroy cumulative vibration from the body of train, and allowing for like vibration by the engine as proposed in formula (1) above, and also impact, it is probable that a factor of safety of 2 might safely be employed, giving 15 600 to be placed opposite the point of intersection of the curves in Plate XXXIX.*

The unit stress for iron in steady tension or compression, is given by Plate XXXIX at 20 000 pounds, 11 400 pounds when repeated from zero to that figure, and 6 600 pounds for vibration with equal tension and compression.

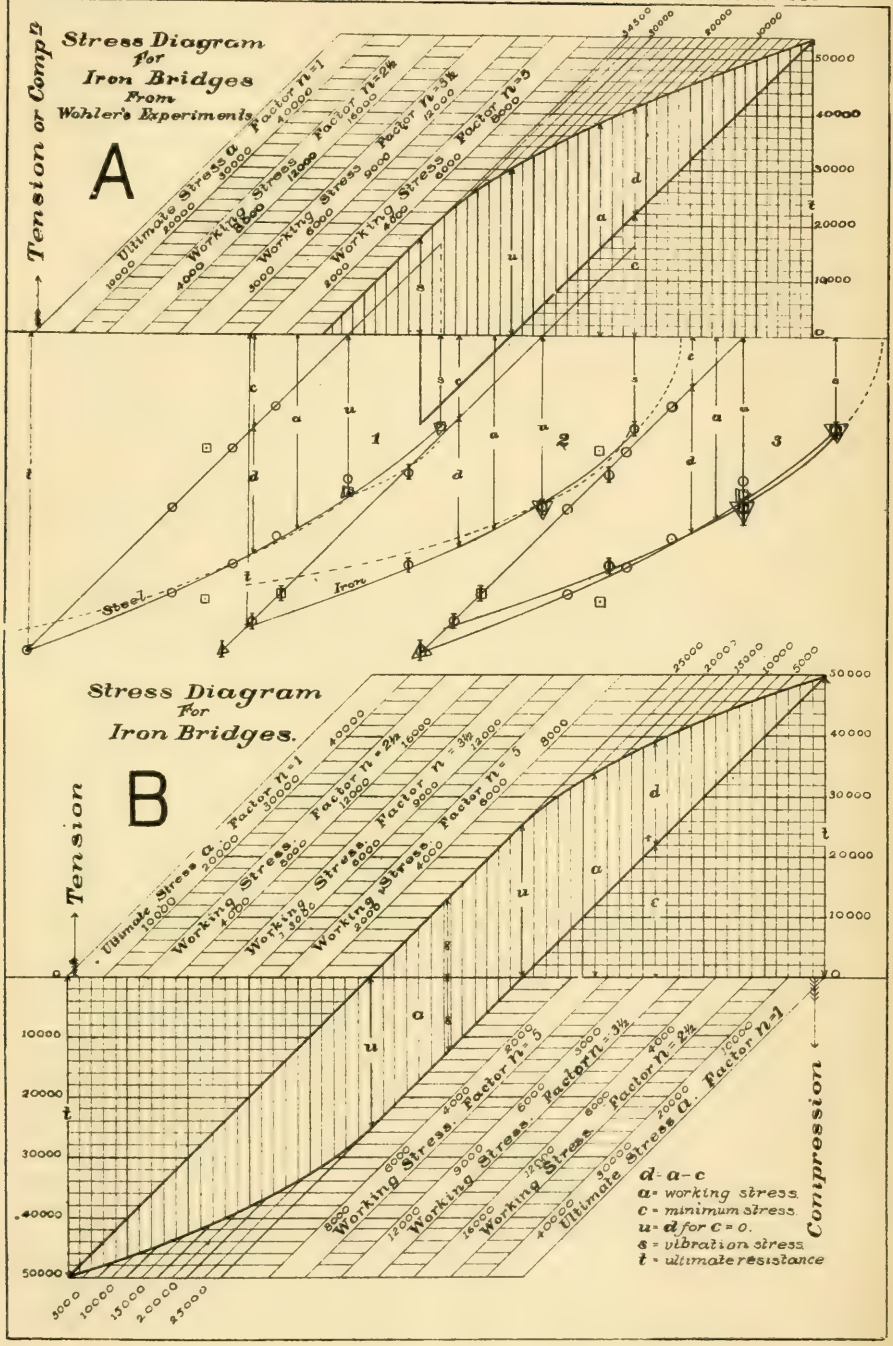
But in drawing the diagram, any scale of equal parts may be put along the line of working stresses according to the judgment of the engineer, and such scale, whatever it be, will give the working stresses in conformity with Wöhler's results; or it may be more convenient to draw several of the diagonal lines on Plate XXXIX, at the junction of each pair of which, on the left side, may be written the factor of safety.

As a practical confirmation of the views given in this discussion, the instance of a certain through iron Howe truss bridge is mentioned, which stood in service about sixteen years, when though yet sound, it was condemned and renewed, on account of dangerous strains in the lower chord, due to placing the floor-beams on the lower chord, two per panel, and each at a distance of 18 inches from the end of panel. Length of truss, about 105 feet; depth of truss, 20½ feet; panels, 13 feet. Aggregate section of lower chords at end panel, 4 inches breadth by 5 inches depth. This was a very peculiar structure, in which all compression members in the main trusses were of cast-iron, and tension members wrought-iron. See a more complete description in Ohio Railroad Report for 1881, p. 289.

The maximum stress in the lower chords figured up at about 25 000 pounds per square inch for an ordinary train and the dead load. See same report, appendix, p. 44, and Van Nostrand's Science Series, No. 60, p. 175.

If a 10 000-pound unit stress calculated for static load, is as high as is safe in some bridges where unaccountable dynamic strains occur to

* This diagram gives only one pair of lines to the factor of safety 2¾ nearly. But the diagram might be full of these pairs of lines, each to a definite factor of safety, all of which factors, 2, 2½, 3, 3½, etc., could be noted at the junction of the respective pairs of lines, making it convenient to use the diagram for any selected factor of safety.


Stress Diagram
For
Iron Bridges
From
Wohler's Experiments.
A
Tension or Compn
Ultimate Stress a
Working Stress
Factor n=1
Factor n=2
Factor n=3
Factor n=5
Steel
Iron
1
2
3
Stress Diagram
For
Iron Bridges.
B
Tension
Compression
d = a - c
a = working stress.
c = minimum stress.
u = d for c = 0.
s = vibration stress.
t = ultimate resistance

carry the stress up to 25 000 or 30 000 pounds, then, if a greater value of calculated strain than 10 000 pounds be found in a particular case like the above, it would seem that the unaccountable part in that case had been low from some cause, so that the sum total of stress should not exceed the 30 000-pound limit.

Hence we inquire whether this bridge was so situated as to be afflicted with cumulative vibration.

As to the panel length, we find the present 13 feet to be an example of the most favorable length, according to figures given above in this discussion; so that it is probable that cumulative vibration never occurred as an effect due to the body of the train.

As to cumulative vibration occasioned by the non-balance of engine drivers, the fact that this bridge was only about 500 feet from a station is against it, as the train would always move so slow as to make this improbable.

These facts, therefore, are in support of the views expressed in the above discussion, viz., that when all strains are accounted for, the working unit stresses may be placed as high as 20 000 or 30 000 pounds with safety.

As a further illustration, the instance of the roof trusses of 180 feet span of a certain Union Depot in the State of Ohio is given. These roof trusses, on inspection, were condemned, as in danger of falling, for strains in the main straining rods greatly in excess of what would in present engineering practice be regarded as admissible.

The strains would be moderate, except for the injudicious and unwarranted splice formed by forging heads upon one side of the rods, which heads were locked one upon another, and banded to prevent slipping off. The bars were rectangular in section. In this way the two bars joined at a splice were out of line, the sides nearest to line offsetting each other by the thickness of the locks. This splice-joint introduced a flexural moment so great, that at the time of the examination the rods near the locks were found so much curved as to be necessarily permanently bent to some extent, because the curvature could not be carried to the extent observed without forcing the iron beyond the elastic limit. The roof has been in service for something like twenty years, with the strains in these rods constantly up about to the elastic limit, as calculation showed; and yet, in spite of all the adverse facts, the roof still stands. The possible strains due to storm and wind dur-

ing the life of the structure are unknown. In some instances snow has been shoveled off to relieve the roof, and it is likely that in a few instances the strains have been 15 to 20 per cent. above the usual and nearly constant strains.

Hence we find here an instance in which strains have been constantly up to nearly 30 000 pounds per square inch for a series of years, far above the 16 000 pounds that might be allowed for a roof, and yet without further signs of failure now than existed ten years ago.

This second instance, therefore, confirms the views stated in this discussion of maximum working stresses, that where all strains are accounted for, the unit working stresses may be carried up to from 20 000 to 25 000 pounds per square inch. This particular roof, however, is not to be regarded as safe, for the reason that there may yet occur such a combination of snow, rain and wind as to cause disaster to the structure by reason of strains greater than those now accounted for.

Finally, to briefly state the points of this discussion, it is seen that I have endeavored to show that this and all railroad bridge specifications should—

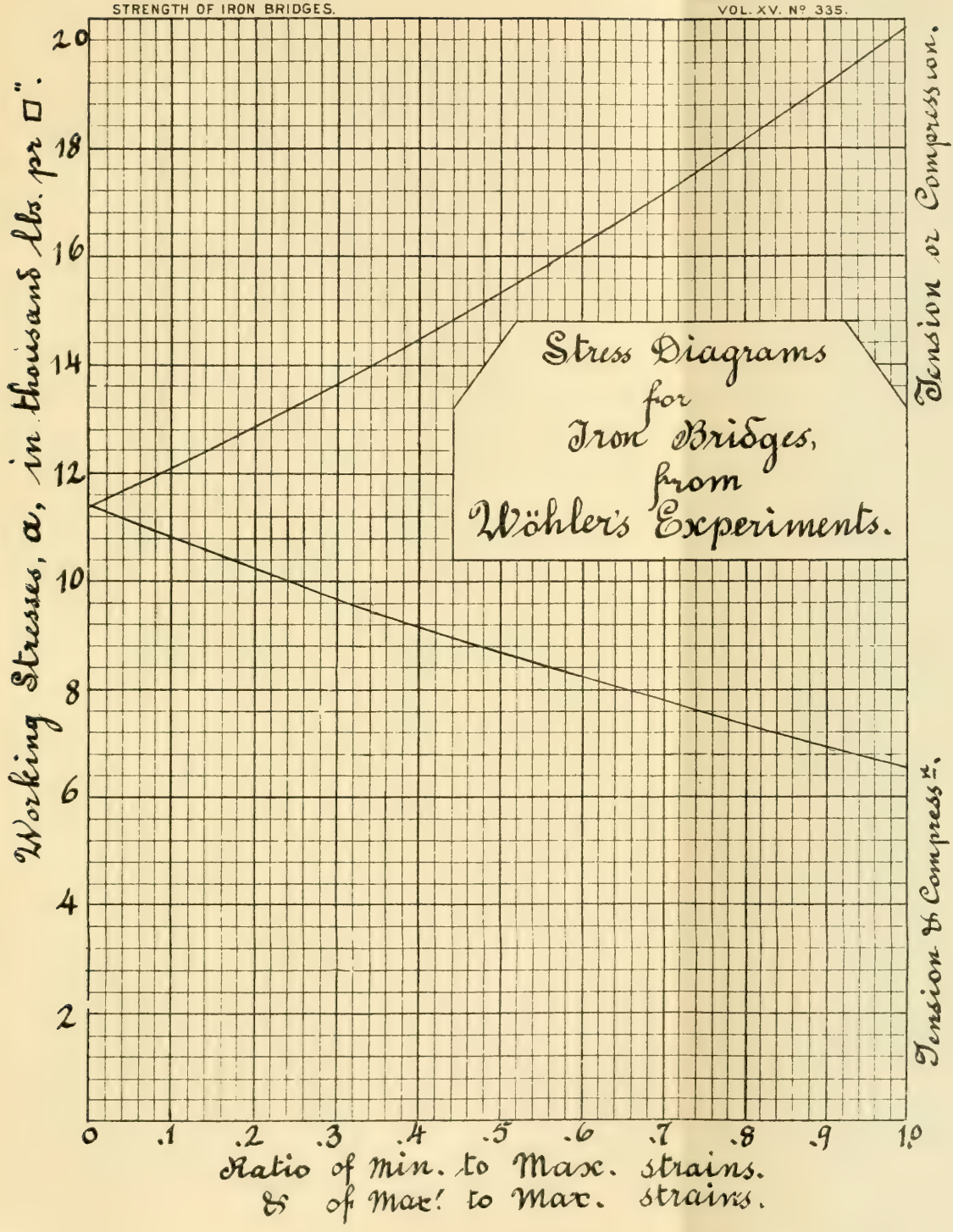
First.—Contain a clause placing the panel lengths of the bridge in discordant relation with the half-car lengths.

Second.—Provide for cumulative vibration due to unbalanced machinery of the locomotive, as in equation (1).

Third.—Add to *second* the proper allowance for impact in estimating live load.

Fourth.—Employ for determining working stresses a diagram like Plate XXXIX, laid out from Plate XXXVIII A, with a factor of safety of about two, or possibly less.

Prof. George L. Vose.—I have examined with care Mr. Wilson's bridge specifications. There can be no question as to the quality of a bridge built in accordance with such a requirement. Certainly no person could be better fitted by long experience to prepare such a paper than Mr. Wilson. I think he has gone very far in this document to do what is so essential, viz., have everything plainly expressed, and nothing left to be inferred. With a model specification, faithfully carried out, two contractors 1 000 miles apart should produce the same bridge. With many of the specifications in use, two contractors in different places would produce very different structures, and both apparently in


Working Stresses, a, in thousands lbs. pr □".
20
18
16
14
12
10
8
6
4
2
Stress Diagrams
for
Iron Bridges,
from
Wöhler's Experiments.
Tension or Compression.
Tension & Compress.
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
Ratio of min. to Max. strains.
& of Max. to Max. strains.

accordance with the same specifications, and both intending to be entirely honest. The more we advance in our knowledge of materials, the harder it seems to be to express our ideas with precision. To say that a bridge should be made with a factor of six throughout, sounds very simple, but I doubt much if any two engineers could agree as to what the factors were in the different parts of a given bridge. I am glad to see that Mr. Wilson does not use this term, which can do little but give false impressions.

To insist in the plainest manner upon the essentials, and to leave no chance for ignorance, dishonesty, or any other form of danger to creep in, is the great point. This Mr. Wilson has certainly done. It may be said that the new specification would not be understood by a board of directors or by the so-called practical man. It is not intended that it should. The specification is for the use of educated engineers who are expert in the principles and the practice of bridge building, as it is only in the hands of such men that the materials of engineering can be safely and economically employed.

J. B. Davis, Jun. Am. Soc. C. E.—I hope S. W. Robinson's column formulas will receive attention in this connection. There is abundant evidence that the end of column formulas is not yet. His are very promising, it seems to me, and are not especially troublesome.

In my own experience in iron truss work, I am accustomed to pack my members, both web and chord, so as to reduce couples about my pins to the shortest of arms, and to bring all my members as nearly as possible into the same planes of stress throughout the whole length of truss. I seek to get these planes continuous throughout the span. To do this I make all my web tension members at least double, which, of course, will be the usual case. I do this however light the work. The result is pairs of members in the same panel. These pairs I require to be matched and marked at the works, and erected by these marks. I also specify a factor of precision for bars of the same pair, and another factor of precision as a limit of extreme deviation between bars of the same specified length in the whole work. Of course this refers to pieces without adjustments. The result, so far, has been quite satisfactory when the trusses are erected and put in "tune," as I call it. I find the bars in the same panel doing the same duty, as nearly as can be ascertained by their apparent tensions.

I consider it very important to study exhaustively how to apply the load to a truss so that no possible expected use of it can fail to convey that load uniformly to its working parts, and distribute it proportionately amongst them. I do not always find it done in existing structures. Likewise I consider the transfer of strains amongst the pieces in just the same light. Everything should be balanced. Of course this is all elementary enough, but is in no danger of being insisted upon too frequently or too forcibly. And this reminds me that a fourteen-year old boy can compute a strain sheet nowadays, but it still takes an engineer to get his data for him, and to use his strains after he has them, and is likely to.

GEO. F. SWAIN, Assoc. Am. Soc. C. E.—The able and interesting paper by Mr. Wilson brings up a number of points which will bear discussion, but I will only touch upon one or two.

In these specifications we find for the first time, to the knowledge of the writer, the formulas of Launhardt and Weyrauch adopted as a basis for the computation of dimensions. But although this may be the first time that any of the newer formulas have been specified, our bridge-builders have not neglected to profit by the experiments of Wöhler, and to adapt their practice to his results. These experiments, and the extended discussions of the past ten or fifteen years, have principally led to a more general recognition and a clearer perception of the principle that the greater the difference between the maximum and minimum loads on a bar, the less should the maximum be. Formerly the so-called factor of safety covered everything beyond the mere statical stresses, including inaccuracies in calculation, effect of repetition of loads, impact, vibration, unequal distribution, possible flaws in the material, deterioration, etc. These experiments have enabled us to take account mathematically of the effect of repetition of loads, but they still leave every engineer to use his own judgment in taking account of the other unknown factors. In allowing for impact and vibration, and inequalities in the distribution of the live load, the writer of the specifications has, it seems to me, not proceeded logically. These factors, depending as they do upon the live load alone, can only be logically allowed for by adding percentages to the static stresses produced by that live load, or to the live loads themselves, as is already so frequently done. If the sudden changes in these percentages be objected to, a sliding scale or a

formula for different spans (as of floor-stringers) may be adopted. If the original form of Launhardt's formula, taking account of repeated applications alone, and not of vibration or impact, be assumed as

$$a = u\left(1 + \frac{\text{min. } B}{2 \text{ max. } B}\right),$$

then the correct way to allow for impact, etc., would seem to be to increase maximum B; not to multiply it by a constant, however, but to increase by a certain percentage that part which is due to the live load. Instead of this, exactly the opposite course is pursued in these specifications; maximum B is diminished by one-half, and then, to make up for this, u, which should have nothing to do with the question, is diminished. Regarding the actual value assumed for u in the formula adopted, it seems to the writer that it might reasonably have been made larger, and that a bar of good wrought-iron could be safely strained to over 15 000 pounds per square inch if the load were perfectly quiescent.

The writer's experience has been that the most convenient method of taking account of Wöhler's experiments, for all cases, is the one given by Winkler, which is really in principle nothing more than that first made use of in practice by Gerber, but recommended by John Griffen and Thomas C. Clarke, Ms. Am. Soc. C. E., in their paper before this Society many years ago, on "Loads and Strains of Bridges." As I do not remember to have seen these formulas quoted in American literature, I may be pardoned for referring to them here. They may be of some interest and novelty to some Members of the Society.

Winkler derives from Wöhler's experiments the following equations, which take account of repeated applications, but not of impact, etc.

Let f = area of cross-section; P max. and P min., the maximum and minimum stresses respectively. Then Winkler finds

$$\text{for wrought-iron} \begin{cases} \text{in tension principally: } f = \dfrac{P\,\text{max.} - 0.45\,P\,\text{min.}}{0.55\,K}; \\ \text{in compr'n principally: } f = \dfrac{P\,\text{max.} - 0.40\,P\,\text{min.}}{0.60\,K_1}; \end{cases}$$

$$\text{for steel} \begin{cases} \text{in tension principally: } f = \dfrac{P\,\text{max.} - 0.56\,P\,\text{min.}}{0.44\,K}; \\ \text{in compr'n principally: } f = \dfrac{P\,\text{max.} - 0.63\,P\,\text{min.}}{0.37\,K_1}; \end{cases}$$

in which K and K_1 are the allowable stresses per square inch for tension and compression respectively, for a purely quiescent load. These may be assumed according to the judgment of the engineer and the

material to be used, but when once chosen, the formulas become very simple. Winkler takes for wrought-iron $K = 20\,000$, and $K_1 = 17\,000$; for steel, $K = 25\,500$, and $K_1 = 31\,000$. Perhaps we should better accord with American practice by assuming for wrought-iron $K = 18\,000$, $K_1 = 16\,000$. We should then have, in round numbers,

$$\text{for tension principally: } f = \frac{P\,\text{max.}}{10\,000} - \frac{P\,\text{min.}}{22\,000};$$

$$\text{for compr'n principally: } f = \frac{P\,\text{max.}}{10\,000} - \frac{P\,\text{min.}}{24\,000}.$$

Suppose, in the case of tension principally, that the dead load causes a tension P_0, and the live load a tension P_1 and a compression P_2; then $P\,\text{max.} = P_0 + P_1$, and $P\,\text{min.} = P_0 - P_2$. Hence the first formula reduces to

$$\text{(tension): } f = \frac{P_0}{18\,000} + \frac{P_1}{10\,000} + \frac{P_2}{22\,000} = \frac{P_0 + 1.8\,P_1 + 0.82\,P_2}{18\,000}.$$

In most cases P_2 will be zero, and we shall have

$$f = \frac{P_0 + 1.8\,P_1}{18\,000}.$$

This method simply amounts, then, to multiplying the live stress by a factor to reduce it to dead, and using 18 000 pounds per square inch as the allowable stress for a quiescent dead load. Winkler allows for impact by multiplying P_1 by a further factor, but he uses simply two factors, one for street and the other for railroad bridges. He would therefore presumably use the same factors for the floor-beam hangers as for the upper chords. This is, of course, incorrect, but the writer believes that by using a scale of percentages to allow for impact, etc., and then applying a formula like the one just given, the demands of modern practice may be met in the simplest and most scientific manner.

The clause in the specification explaining the mode of calculating the upper chord when the load rests upon it between the joints, appears rather complicated, and it seems to the writer that it would be just as well, in cases where this unfavorable arrangement is necessary, to proportion the chord for the direct thrust and the bending, considering the chord-piece as a supported beam simply; especially in view of the uncertainty attending the ordinary method of calculating pieces for compression and flexure.

These specifications require that the flanges of plate girders shall be calculated to bear the entire moment, and the web the entire shear. The writer is aware that this is the general practice, but he has never been

able to see the slightest reason, in this case, for taking things other than they actually are. The fact is, that the web does bear part of the moment; and if the factor of safety to be used, the allowance for impact, the effect of repetition, etc., be rationally taken into account, why should any further allowances be made? The writer sees, every day, girders in which the moment borne by the web has probably been neglected, and in which the web-splice has evidently been also proportioned to bear the shear alone, thereby no doubt putting an excessive strain on the rivets in the splice, and making a girder which is not of equal strength throughout. As the web really does bear part of the moment, it seems only reasonable that the splice should be calculated for it, and why not the rest of the girder also?

MACE MOULTON, M. Am. Soc. C. E.—Some two years ago I had occasion to use a modification of Launhardt's formula in proportioning the members of several spans of varying lengths.

The differences in the allowable unit strains from those I had previously used were so great, that I concluded to investigate, in a general way, the working values as given by Launhardt's formula applicable to a large range of spans, and also to discuss the formula in connection with the records of the experiments on which it was founded.

The results, as to unit strains and factors of safety which were obtained, are shown in the table on page 450.

The comparison was for simplicity confined to tension members in bottom chord, and for compression to the end posts. This was easily made by assuming that the strains in center bottom chord and in end posts were directly proportional to the total loads on span when fully loaded. Hence for any span:

$$\frac{\text{Min. } B.}{\text{Max. } B.} = \frac{\text{Total dead load.}}{\text{Total dead} + \text{total live.}}$$

The engines and train assumed were same as specified by Mr. Wilson, equated for each span. The weights of spans were from data at hand, partly actual and partly calculated for the purpose, closely enough for purposes of comparison.

The results of Launhardt's formula in tension are shown in column 4, and in compression in column 12. To tabulate results of common practice, with allowances for impact and influence of increased percentage of live load as compared with dead, columns 1 and 2 for tension, and 10 for compression were formed.

Length of Span.		For Tension.		For Tension.		For Tension.		For Tension.		For Comp.		For Comp.		For Comp.		For Comp.	
		10 000 pounds per Square Inch, With Impact Deducted.		Launhardt's Formula: $v = u\left(1 + \frac{\text{min. } B}{2\,\text{max. } B}\right)$ $u = \frac{30\,000}{3} = 10\,000$		Factor 4 for Dead. Factor 6 for Live. $a = 12\,000\left(-4\,000 \div \frac{\text{live}}{\text{live}+\text{dead}}\right)$		Wilson's Formula: $a = u\left(1 + \frac{\text{min. } B}{\text{max. } B}\right)$ $u = 7\,500$ for Shapes. $u = 7\,500$ for Double Riefined Iron.		8 000 pounds per Square Inch, with Impact Deducted.		Launhardt's Formula: $a = u\left(1 + \frac{\text{min. } B}{2\,\text{max. } B}\right)$ $u = 10\,000$ pounds.		Factor 4 for Dead. Factor 6 for Live. $a = 10\,000\left(-3\,300 + \frac{\text{live}}{\text{live}+\text{dead}}\right)$		Wilson's Formula: $a = u\left(1 + \frac{\text{min. } B}{\text{max. } B}\right)$ $u = 6\,500$ pounds.	
Feet.	Impct pr. ct.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
		Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.
20	35	7 600	6.	10 500	4.6	8 400	5.7	7 700	6.	6 080	6.6	10 500	3.8	7 030	5.7	7 150	5.6
30	30	7 940	5.8	10 650	4.5	8 520	5.6	7 910	5.8	6 350	6.3	10 650	3.7	7 129	5.6	7 350	5.4
40	25	8 200	5.6	10 800	4.4	8 640	5.6	8 120	5.7	6 540	6.1	10 800	3.7	7 230	5.5	7 540	5.3
50	22.5	8 400	5.5	10 900	4.4	8 720	5.5	8 260	5.6	6 720	6.	10 900	3.7	7 290	5.5	7 670	5.2
60	20	8 600	5.4	11 000	4.4	8 800	5.5	8 400	5.5	6 890	5.8	11 000	3.6	7 360	5.4	7 800	5.1
70	17.5	8 800	5.2	11 100	4.3	8 880	5.4	8 540	5.4	7 040	5.7	11 100	3.6	7 430	5.4	7 930	5.
80	15	9 000	5.6	11 200	4.3	8 920	5.4	9 320	5.4	7 170	5.6	11 200	3.6	7 460	5.4	8 060	5.
100	10	9 300	5.4	11 300	4.3	9 080	5.3	9 450	5.3	7 450	5.4	11 300	3.5	7 590	5.3	8 190	4.9
130	4	9 750	5.1	11 400	4.2	9 120	5.3	9 600	5.2	7 780	5.1	11 400	3.5	7 620	5.2	8 320	4.8
150	0	10 000	5.	11 500	4.2	9 200	5.2	9 750	5.1	8 000	5.	11 500	3.5	7 690	5.2	8 450	4.7
175	0	10 000	5.	11 700	4.1	9 320	5.2	10 050	5.	8 000	5.	11 700	3.4	7 790	5.1	8 710	4.6
200	0	10 000	5.	11 850	4.1	9 480	5.1	10 275	4.9	8 000	5.	11 850	3.4	7 920	5.	8 905	4.5
250	0	10 000	5.	12 250	3.9	9 760	4.9	10 875	4.6	8 000	5.	12 250	3.3	8 150	4.9	9 425	4.2
300	0	10 000	5.	12 350	3.9	10 080	4.8	11 025	4.6	8 000	5.	12 350	3.2	8 420	4.7	9 555	4.2
350	0	10 000	5.	12 550	3.8	10 320	4.7	11 325	4.4	8 000	5.	12 550	3.2	8 610	4.6	9 815	4.1
400	0	10 000	5.	13 100	3.7	10 480	4.6	12 150	4.1	8 000	5.	13 100	3.1	8 750	4.6	10 530	3.8
450	0	10 000	5.	13 400	3.6	10 720	4.5	12 600	4.	8 000	5.	13 400	3.	8 940	4.5	10 920	3.7
500	0	10 000	5.	13 500	3.6	10 840	4.4	12 750	3.9	8 000	5.	13 500	3.	9 040	4.4	11 050	3.6

The unit strains there shown are the equivalent strains, starting with a basis of 10 000 pounds per square inch in tension and 8 000 pounds in compression, which would give the same sectional area as if the impact percentages were added to the live load.

I then attempted to formulate the common practice ignoring impact, combined with a recognition of Wöhler's law, using the factors of 4 for all dead load and 6 for all live load.

The results of this are shown in columns 6 and 14, where general ultimates of 48 000 and 40 000 pounds per square inch were assumed. Columns 5 and 7 were also formed on the basis of an ultimate strength of 48 000 pounds and Wöhler's law, that rupture of material may be caused by stresses from repeated applications of loads, none of which equal the breaking load. The differences of these stresses being measures of the strength of the material, has, I believe, been fully demonstrated by his own and Spangenberg's experiments on small pieces, as well as those by Fairbairn on riveted girders.

These experiments showed that near the elastic limit a certain variable load was one and a half to twice as severe in its effect as the same load when quiescent.

From this it became apparent that engineers should take into account the effect of moving loads on a structure as distinguished from dead loads.

That this has been done, and that it enters to a certain degree into the every-day practice of American engineers, is evinced by the quotations in Mr. Wilson's paper.

Since reading his paper I have continued the comparison to include results from proposed formula. These are shown in columns 8 and 16.

I have also determined the factors of safety shown in columns 3, 9, 11, 17 referred for tension to ultimate strength specified, viz.: 46 000 pounds per square inch for plates and shapes (used up to 80 feet span), and 50 000 pounds for eye-bar material. All factors for compression are referred to 40 000 pounds per square inch ultimate strength.

The question which arises in the examination of the foregoing table, is whether the practice as generally pursued carries the principle of Wöhler through consistently, and whether the formulas and constants given by Mr. Wilson give more rational results than those heretofore used.

Launhardt's formula, using his divisor of 3 for working strength,

averages about a factor of 3 for dead and 5 for live loads, while that used in specifications amounts to using factor of 3 for dead and 6 for all live. The present general practice more nearly approaches factors of 4 and 6 for dead and live respectively, all referred to the ultimate strength of the material.

If consistently carried out, the latter would be as before stated, about as shown in columns 6, 7 and 14, 15. If no allowance for impact, or the principle enunciated by Wöhler was made, the uniform factor would, of course, be 5.

Study of the results of experiments before mentioned did not to me seem to wholly justify the use of Launhardt's formulas as his and Weyrauch's constants left it, neither did the common practice seem to sufficiently comply with the law of variations in effect of live and dead loads. Nevertheless the latter was certainly safe within limits discussed.

The formula proposed, however, seems to me to furnish a close approximation for tensile stress to the conditions imposed by the loads. It assumes that the effect of live load is twice as injurious as that of dead load.

This was the extreme limit of variation found by experiment near the elastic limit of the material, and therefore we have a corresponding and additional margin of safety for strains never exceeding one-half the elastic limit.

For compressive strains I should consider it equally good for upper flanges of well-braced plate or lattice girders under 80 feet span.

In its application to the constants of the well known column formulas, we have, however, a different case to consider.

These formulas, as adopted by Mr. Wilson, with working strain of 8 000 pounds per square inch, decreased in proportion of length to least radius of gyration, give results agreeing as closely as any with those of testing machine on square columns.

The application of the loads applied in the testing machine being gradual, the post may be assumed to be in a condition similar to that in the structure with a gradual livel oad coming on, without shock, of a sufficient intensity to cause rupture.

Taking the specified formula for permissible working stress for all live loads we have $a = 6\ 500$ pounds per square inch, which corresponds as to conditions with the 8 000 pounds in column formulas, and which gives a factor of 6 as referred to 40 000 pounds ultimate strength per square inch in compression.

It has been proven by a multitude of experiments, that when no lateral flexure occurs, the rupture of any structural material, subjected to bending stress, occurs on the side which is strained in tension. From this fact, it is fair to assume that when the column is properly stayed to prevent flexure from loads producing stresses in the material up to the elastic limit, the material composing the column is as well able to take compressive as tensile stress.

Of course, in practice, our columns are designed to admit working stresses only which are far within the elastic limit. Hence, it seems rational to allow a working value equal to that for tension in the same material.

Mr. Wilson's constant is, for compression, less than that for tension, hence, still farther on the safe side.

I will state that the percentages added to live load in table, under head of impact, were obtained by using data afforded by Mr. B. Baker, which he deduced from experiments by Baron Von Weber, and applied on the heaviest English locomotives. Mr. Baker allowed a percentage for impact for short spans to be added to the live load, and also decreased his unit strain.

Percentages given simply combine these operations, and are applicable to any other live loads, as well as those considered by Mr. Baker.

A. P. Boller, M. Am. Soc. C. E.—In commenting upon Mr. Wilson's paper, I recognize the fact of his unusual opportunities for noting the behavior of iron in structures under the wear and tear of actual service; opportunities denied the regular professional bridge-builder, who rarely sees his work after it is once taken off his hands, unless he makes a special mission for the purpose. The first point brought out by Mr. Wilson is the purely arbitrary practice among engineers in their selection of what may be termed the base strain for proportioning structural work, and his recommendation of the Launhardt and Weyrauch formula is in the direction of uniformity of practice. No accurate formula can ever be devised, any such being as arbitrary as methods in common use; but this one of Launhardt's seems to crystallize in a simple way the recognized truths, that the greater the ratio between dead and moving load, the less should material be strained; and that where material is subject to alternate tension and compression, it should be proportioned with a unit strain largely in excess of what would be required

for all tension or all compression. The formula is certainly on the side of safety, having been worked out from deductions based upon impact experiments, involving conditions to which few, if any, bridge structures are actually subjected. Impacts continuously kept up to rupture, on comparatively small pieces, create a ceaseless molecular vibration that even our elevated railways are not subjected to; and it seems to me going to the extreme of conservatism to judge of the value of iron construction wholly on the same basis. It is impossible for the engineer to be bound entirely by an empirical formula, no matter how convenient its use, as varying circumstances will control his judgment as to permissible strains. A bridge structure infrequently used, or one which, in the nature of things, can never be subjected to a high-speed traffic, may safely be proportioned for a higher unit strain than structures subjected to great speed or in frequent use. Why Mr. Wilson should assign a less compressive than tensive value to wrought-iron is not clear, and this is hardly in accord with modern experimental knowledge. The elastic limit in compression is fully as high as in tension, and the ultimate compressive strength, where bending is not in question, is at least equal to the highest tension value, and in large sections certainly higher. Then why make the distinction ?

I question very much Mr. Wilson's assumption that continuous compression members over points of support are substantially hinged columns, and should be so treated. This is an important fact, if true, as it would add a large percentage of metal to upper chord members as usually proportioned. It seems to me that such a chord is held as securely by the pins passing through it as if riveted by gusset plates, or supported direct on post heads, and that the assumption there is a tendency for compound bending is an over-refinement that has very serious tendencies, not only adding quite perceptibly to the amount of material in a given structure, but also on the practical questions involved in long spans. There is one point in the specifications that will, I think, strike many engineers as exceedingly critical of common practice, and that is the condemnation of plate girders having no upper flange plates. I am sure most engineers will agree that any exact theoretical solution of the plate-girder problem is impossible, and that the fewer the parts of which such a girder is composed, the more nearly can theory and practice be made to harmonize. Consequently, so far from not insisting upon plating the flanges, it is desirable to retain but a pair of

angles for each flange, as far as practicable, even at the expense of more metal, and only have recourse to plating when constructive necessity demands it; and even in that case not to build up theoretical sections with too many thin plates. Piling up thin plates for the sake of economy of metal can only be done at the sacrifice of the economy of the shop. I know of no branch of constructive iron-work that demands more sound, common sense than plate-girder work, in which simplicity and fewness of integral parts are base principles.

Mansfield Merriman, M. Am. Soc. C. E.—The method of assigning the working stresses per square inch for members subject to alternating stresses, by means of the formulas of Launhardt and Weyrauch, appears more rational and satisfactory than the use of percentages varying with length of span or with the ratios of dead to live load. It is, however, an objection to Weyrauch's formula that it does not contain t, the ultimate strength of the material under a static stress. Launhardt's formula contains t, but the vibration strength s is absent, and this would probably affect the value of the working stress a for a small range of stress near the zero point.

Let R denote the ratio of the least limiting stress in the member to the greatest limiting stress. If these limiting stresses are both tension or both compression, R will be positive, but if one is tension and the other compression, R is negative. Then the formulas of Launhardt and Weyrauch may be written

$$a = u\left(1 + \frac{t-u}{u} R\right) \text{ for positive } R.$$

$$a = u\left(1 + \frac{u-s}{u} R\right) \text{ for negative } R.$$

Here there are two formulas to express the variation of a as R varies from $+1$ to -1, whereas it would seem that one formula expressing a as a continuous function of R in terms of t, u, and s would be a more rational expression of the true law.

The following formula, whose deduction is elsewhere given, contains u, t, and s, and gives values of a corresponding to values of R from $+1$ to -1:

$$a = u\left(1 + \frac{t-s}{2u} R + \frac{t+s-2u}{2u} R^2\right) \quad \ldots\ldots\ldots\ldots (1)$$

This formula satisfies the three limiting conditions; thus, if $R = +1$,

the value of a is t; if $R=0$, the value of a is u; and if $R=-1$, the value of a is s; and these are the three values indicated by the experiments for a great number of repetitions of stress between the limiting values.

For wrought-iron, experiments show that $t=2u$ and $s=\frac{1}{2}u$. Hence formula (1) becomes

$$a=u\left(1+\frac{3}{4}R+\frac{1}{4}R^2\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

To use this formula for determining values of the working unit stress a, the value of u must be taken as one-half of the working strength per square inch for a static load liable to no variation. For example: If the working strength under a static stress be taken as 14 000 pounds per square inch, the formula is

$$a=7\,000\left(1+\frac{3}{4}R+\frac{1}{4}R^2\right)$$

Now for a member whose stress ranges from 40 000 pounds tension to 100 000 pounds tension, the value of R is $+\frac{4}{10}$ and that of a is 9 380 pounds per square inch; but for a member whose stress ranges from 40 000 pounds compression to 100 000 pounds tension, the value of R is $-\frac{4}{10}$ and the value of a becomes 5 180 pounds per square inch.

On Plate XL are plotted the curves corresponding to formula (2) for the three cases, $u=7\,500$, $u=7\,000$, and $u=6\,500$ pounds per square inch. The straight lines corresponding to the formulas of Launhardt and Weyrauch are also drawn for the case of $u=7\,500$. The diagram shows that the formula (1) agrees with those of Launhardt and Weyrauch when $R=+1$, $R=0$, and $R=-1$; but that for all other values of R, it gives somewhat smaller values of the allowable working unit stress a. Launhardt's formula is that of a straight line, Weyrauch's formula is also a straight line intersecting the first, while (1) is the equation of a parabola passing through the three points best determined by experiments.

The very few experiments made by Wöhler for values of R between $+1$ and 0 seem to be somewhat better satisfied by Launhardt's formula than by formula (1). No experiments were made for values of R between 0 and -1.

The following table gives values of a corresponding to the three curves shown in the diagram:

	($u = 7\,500$)	($u = 7\,000$)	($u = 6\,500$)
R	a	a	a
+1.0	15 000	14 000	13 000
+0.9	14 081	13 143	12 204
+0.8	13 200	12 320	11 440
+0.7	12 356	11 533	10 709
+0.6	11 550	10 780	10 010
+0.5	10 781	10 063	9 344
+0.4	10 050	9 380	8 710
+0.3	9 356	8 733	8 109
+0.2	8 700	8 120	7 540
+0.1	8 081	7 543	7 004
0.0	7 500	7 000	6 500
—0.1	6 956	6 493	6 029
—0.2	6 450	6 020	5 590
—0.3	5 981	5 582	5 184
—0.4	5 550	5 180	4 810
—0.5	5 156	4 813	4 469
—0.6	4 800	4 480	4 160
—0.7	4 481	4 183	3 884
—0.8	4 200	3 920	3 640
—0.9	3 956	3 693	3 429
—1.0	3 750	3 500	3 250

The value of a may be expressed also in the following manner:

$$a = p\,S \qquad (3)$$

in which S is the working strength $2\,u$ for a purely static stress, and p is a number less than unity or the percentage to be taken of S for a particular value of R. By comparison with formula (2) it is seen that the value of p is

$$p = \frac{1}{2}\left(1 + \frac{3}{4}R + R^2\right)$$

The following table gives values of the percentage p for different values of R. When R has been found for any particular case p may be taken from this table, and then the permissible working unit-stress a is p times the greatest allowable working unit-stress S.

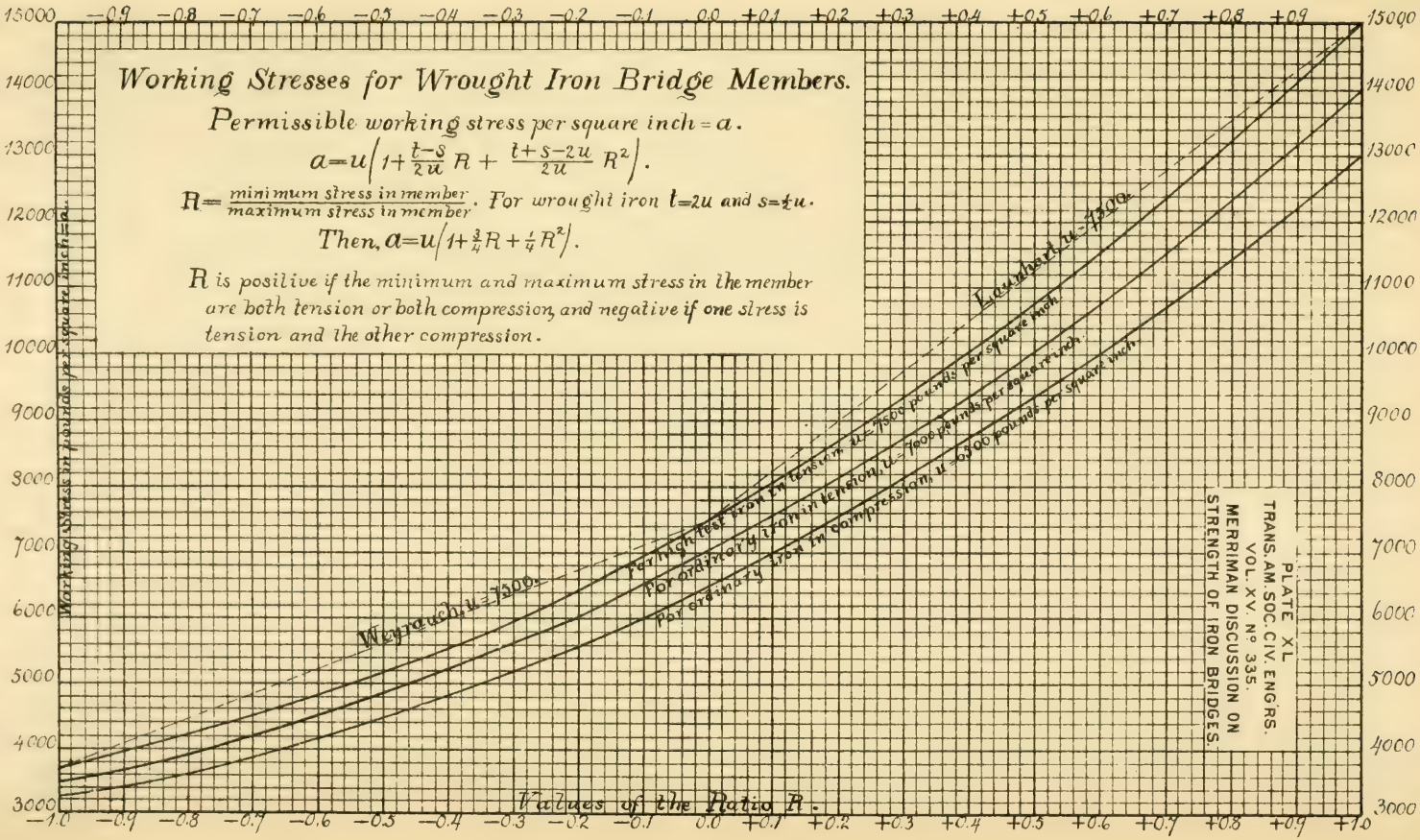
R	p	R	p
+1.0	1.000	+0.4	0.67
+0.9	0.939	+0.3	0.624
+0.8	0.88	+0.2	0.58
+0.7	0.824	+0.1	0.539
+0.6	0.77	0.0	0.50
+0.5	0.719		

R	p	R	p
0.0	0.50	—0.6	0.32
—0.1	0.464	—0.7	0.299
—0.2	0.43	—0.8	0.28
—0.3	0.399	—0.9	0.264
—0.4	0.37	—1.0	0.25
—0.5	0.344		

JAMES G. DAGRON, C. E.—The author, in his specification requires that the eyes on all tensile members shall have fifty (50) per cent. excess of material at the pin when the diameter of pin does not exceed the width of bar, and one hundred (100) per cent. excess when the diameter is twice the width of bar or over; for intermediate sizes of pins the excess of eye to be made proportional to their diameter. An additional clause specifies that all links and rods, if tested to destruction, shall part through the body and not through the head or pin-hole.

These requirements do not agree with the results of many tests made by myself, during the past year, on eye-bars manufactured by three of the leading bridge companies; in fact these experiments have shown that the best results are obtained when the diameter of the pin is equal to or greater than the width of the bar, provided proper care is taken in the manufacture of the head. The proper amount of excess to be given in the latter case has yet to be determined, but I think it should not be greater than when the diameter of the pin is less than the width of the bar. It is my opinion, however, that it would be injudicious to reduce the requirements of the specifications to the percentages of excess claimed by certain manufacturers, these claims being based upon a few exceptional tests such as those cited by the author in his paper, and there is nothing to prove that they can be obtained in a current manufacture, for it should not be forgotten that these special test-bars are very liable to be the object of much greater care than bars of a current fabrication would receive.

The percentage of bars which break through the head or pin-hole when tested to destruction, is much greater than would be supposed. It is true, however, that a certain proportion of these breakages may be chargeable to improper proportioning of width of bar for a given diameter of pin. Of the 125 bars tested by me, fifty-two (52) were of steel and the balance iron. The steel bars ranged in width from 3 to 7 inches, the smallest section being 3 by $\frac{13}{16}$ inches and largest 7 by $1\frac{1}{8}$ inches. The diameter of the pins varied from $4\frac{1}{8}$ to $6\frac{7}{8}$ inches, and in the


Working Stresses for Wrought Iron Bridge Members.
Permissible working stress per square inch = a.
a = u / (1 + (t − s)/2u R + (t + s − 2u)/2u R²).
R = minimum stress in member / maximum stress in member. For wrought iron t = 2u and s = ½u.
Then, a = u / (1 + ¾R + ¼R²).
R is positive if the minimum and maximum stress in the member are both tension or both compression, and negative if one stress is tension and the other compression.
Working Stress in pounds per square inch = a.
Values of the Ratio R.
Wrought, u = 7500.
For high fibre in tension, u = 7500 pounds per square inch.
For ordinary iron in tension, u = 7000 pounds per square inch.
For ordinary iron in compression, u = 6300 pounds per square inch.
Lucomhard, u = 7500.
PLATE XL
TRANS. AM. SOC. C.IV. ENG'RS.
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STRENGTH OF IRON BRIDGES.

majority of cases was greater than the width of bar. The iron bars broken were mostly five (5) and six (6) inches in width, the pins in the majority of cases being of a diameter less than the width of bar. The percentages of excess of material through the pin-hole varied from fifty (50) to sixty (60) per cent. for both steel and iron bars. The majority of the steel bars were made by the hydraulic upsetting and forging process, and notwithstanding the care taken in their manufacture, nearly twenty-nine (29) per cent. of the total number tested broke through the pin-hole or in the head, and of these over fifty-seven (57) per cent. had pins of a greater diameter than the width of the bar; the excess of material through the pin-hole in every case being about fifty (50) per cent.

The iron bars were mostly made by upsetting and piling, and in this case nearly forty-four (44) per cent. broke through the pin-hole or in the head. The excess of material through the pin-hole was in every case greater than fifty (50) per cent.

The bars broken were not special test bars made for the occasion, but were taken from the piles of bars intended for use in the different structures, and can thus be fairly supposed to represent the present stage of eye-bar manufacture.

The results obtained have shown that there is considerable room for improvement in this line, and that until it is conclusively proven that this improvement has taken place it would be unwise to reduce the percentages of excess, especially in the case of steel bars, where so many additional factors, such as proper heating and annealing, tend to complicate the problem of successful manufacture.

William Sellers, M. Am. Soc. C. E.—The paper concludes with the statement: "Great care has been taken in framing the conditions for quality of material and workmanship to insure having what is desired, and what the present advanced state of manufacture can produce, without asking for what is impossible to obtain except at extraordinary cost." I propose to confine my discussion of this paper to that portion of the specification thus referred to, believing that, in the present state of the art, less difference of opinion exists as to the strains and the methods of computing them, than as to the details of construction and the quality of material and workmanship. And at the outset I have to say that the author is evidently sincere in his desire not to burden his specifications

with requirements that cannot be complied with, except at an extraordinary cost, and in the main he has been successful. But there may be requirements, the fulfillment of which cannot be determined by the physical tests prescribed; that nothing, in short, but the personal supervision of the engineer, or his inspector, at every stage of the manufacture could enforce, and as this last is practically impossible, he finally contents himself with the behavior of his material under his physical tests. Now the question arises: Is it wise to retain provisions that never are and never can be enforced? I allude to the clause "must be double-rolled after and directly from the muck bar (no scrap will be allowed)." To comply with this provision literally would certainly involve additional cost, whereas, with rare exceptions, no extra charge is made by the manufacturer when this provision, common to most specifications, is brought to his notice. There are exceptions, who say they will agree to comply with the physical tests, but they cannot conscientiously say they will comply with this provision of manufacture. I believe it is as unwise as it is unfair to prescribe conditions for manufacturing which practically prevent such men from furnishing this class of material, and it would be well for the engineer not only to limit his requirements to what is desired, but also to determine what means are at his disposal for insuring that he will obtain what he desires, and then limit his requirements strictly to his means for insuring their fulfillment. In framing specifications it must be remembered that they are to be construed by inspectors who are generally innocent of any practical knowledge of the material with which they are dealing, or of modes of construction, or of the means for its accomplishment; and too much care cannot be taken to avoid ambiguities and the exercise of their judgment. And to this end I suggest that the words "hereinafter described" should be inserted after test piece in the first, and "in addition to the first requirements" should be appended to the second, third and fourth paragraphs of Quality of Materials.

The test piece as described contemplates a uniform length with varying thicknesses. It is desirable that the ductility should be comparable in all test pieces; and this cannot be done unless the length is a uniform multiple of their shortest transverse section. It is impossible to elucidate this proposition within the limits of this discussion, but I may refer to a paper by W. Hackney, read before the Society of Civil Engineers in London, and published in Vol. 76, Session 1883–84, Part II, in which the advantages of this modification are clearly set forth.

Under Workmanship an omission occurs which I believe has escaped the notice of the author. It should read, "In the case where rough bolts are permitted a variation of one-sixteenth of an inch will be allowed between diameter of bolt and hole."

Since the report of tests on steel eye-bars was made which the author has quoted, the Edge Moor Iron Company have nearly completed the modification of their plant for the manufacture of such bars, but I am unable as yet to submit any later data than that contained in the paper. It is to be regretted that the author has limited his specifications to the use of wrought-iron only, for at the present day steel enters largely into the composition of nearly all bridges, and his formulas are as well adapted to the one material as to the other. So that it only requires that the requisite tests should be prescribed to obtain the quality with reasonable exactitude, without resorting to tests that add materially to the cost, while they add literally nothing to the security of the structure, and I suggest that the discussion might be profitably extended to this branch of the subject.

E. Thacher, M. Am. Soc. C. E.—Although I do not fully agree with the author in all particulars, I think his specification is a move in the right direction, and that it will give safe and fairly economical results. From the standpoint of a bridge engineer it is too complex; too much time is required in estimating; and essentially the same results can be obtained by simpler means. The specification gives three different distributions of rolling load, the maximum of these being used in proportioning every member of the structure. Now, for any length of span or class of engine, a uniformly distributed load can be found, which, when used in calculation, will give essentially the same results as the wheel loads; and by aid of preliminary moment tables these maximum equivalent loads can be readily calculated and tabulated once for all, and the various competitors for work would be relieved from the necessity of devoting hours of time to calculations which could be made as well in a few minutes; and as the loads are typical at best, great exactness in this regard does not appear to me essential. Having fixed upon the loads, I would have the stresses correct for those loads, which is not the case if the cross-girder load under the drivers is considered the head of the train, as the author recommends. To be sure it is on the side of safety, but not consistently so, as the percentage of gain varies largely in the different members.

The division of dead load at panel points, as specified, requires too much labor, and it cannot well be observed except by revision. My practice has been to consider two-thirds panel weight at loaded and one-third at unloaded apex, which is believed to be sufficiently correct for all practical purposes.

I have confidence in the correctness of Launhardt and Weyrauch's formulas. The ordinary arbitrary provisions for repetitions of load and impact are inconsistent and unsatisfactory. The modification of Launhardt's formula adopted by the author, allows $33\frac{1}{3}$ per cent. as the maximum effect of impact alone, which appears to me about right. Ordinarily the track on bridges is the best on the road, and under these circumstances the effect of impact from various recorded experiments appears to be next to nothing; but to provide for imperfections and to a certain extent, accidents, it is well to make a liberal allowance.

The formulas of Launhardt and Weyrauch allow for the fatigue of the material, but I can see no good reason why their results should be used in Rankine's formulas for struts which fail partially or wholly by flexure; nor, in fact, much necessity for Rankine's formulas. Iron struts deflect very little until near the breaking point, and repetitions of stress within safe or working limits are not repetitions of flexure. If Rankine's formulas are used, I believe the numerator should be constant or nearly so, and that the resulting working stress should not exceed the allowed stress by Launhardt's formula.

For members subject both to tension and compression, I can see no relation between the tension to which it is subjected and its failure by flexure. I believe flexure formulas should consider compression only, but that the resulting working stress should not exceed the amount given by Weyrauch's formula for alternate stress.

Plates XXXV and XXXVI, comparing the results of experiments with various proposed formulas, and Plate XXXVII, comparing proposed formulas with each other, are very interesting and instructive. So long, however, as there remains such a wide discrepancy in opinion as to the proper value of the constants that should be used in Rankine's formulas, as illustrated by Plate XXXVII, resulting in a variation of from 60 to upwards of 100 per cent., it would appear to me that the formulas are of very little service. If on Plate XXXV we draw a line connecting the maximum values of a, and another connecting the minimum values, leaving out of consideration welded

tubes and swelled octagon columns, there will be inclosed an area showing the variations in the results of experiments on flat-ended struts. If now a line is drawn, starting at 45 000 at $\frac{l}{r} = 0$, and ending at 9 000 at $\frac{l}{r} = 240$, it will pass very nearly through the center of this area, and will apparently represent more nearly than any of the proposed curved lines, the average strength of the struts. The equation of this line is $a = 45\,000 - 150\,\frac{l}{r}$ or allowing a factor of safety of 5, $a = 9\,000 - 30\,\frac{l}{r}$ which is a simple formula, easily applied. Likewise, if on Plate XXXVI we draw lines connecting the maximum and also the minimum values of a, leaving out of consideration a few scattering results, due probably to exceptional character of material or manner of conducting the experiments, there will be inclosed an area showing the variations in the results of experiments on pin-ended struts. If now a line is drawn, starting at 45 000 at $\frac{l}{r} = 0$, and ending at 10 000 at $\frac{l}{r} = 175$, it will pass nearly through the center of this area, and will represent quite as nearly as any of the proposed curved lines the average strength of the struts. The equation of this line is $a = 45\,000 - 200\,\frac{l}{r}$ or, allowing a factor of safety of 5, $a = 9\,000 - 40\,\frac{l}{r}$ which is also a simple formula easily applied. To sum up, I would propose the following formulas for working stresses:

TENSION ONLY.

$$a = 7\,500\left(1 + \frac{\text{min. stress in member}}{\text{max. stress in member}}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

COMPRESSION ONLY.

$$\left.\begin{aligned} b &= \left(9\,000 - 30\,\frac{l}{r}\right) \text{ flat ends} \\ b &= \left(9\,000 - 35\,\frac{l}{r}\right) \text{ one flat and one pin end} \\ b &= \left(9\,000 - 40\,\frac{l}{r}\right) \text{ pin ends} \end{aligned}\right\} \text{ Not to exceed } a \ldots\ldots\ldots (2)$$

ALTERNATE TENSION AND COMPRESSION.

For compression only, see formula (2).

For greater stress tension or compression

$$c = 7\,500 \left(1 - \frac{\text{max. stress of lesser kind}}{2 \text{ max. stress of greater kind}} \right)$$

use the one giving the greatest area of section.

I do not think it necessary that horizontal struts, continuous over points of support, should be considered as hinged at such points, as undoubtedly the weight of the piece is more than sufficient to overcome any tendency to bend upwards; but for members vertical, or nearly so, with intermediate points of support, such as posts and bridges in viaducts, I fully agree with the author.

I think it is very essential that the center of pin should always be placed as nearly as possible in the neutral axes of the section. Too little attention is usually paid to this. No provision can be made for taking up eccentric stresses except by large additions to the section. It is very common in the design of bridges to have the top chord and end braces consist of two rolled or riveted channels with equal flanges and a top plate, the center of pin being placed at the center of the channels. This plate is always counted as effective area, notwithstanding its addition is often a positive injury, the section being stronger without than with it.

I do not approve the method specified by the author for proportioning the top chords of deck bridges when the floor system rests directly on them. Such construction I consider poor, at best, and prefer not to use it if it can be avoided. The stresses are much affected by deflections at panel points, character of splices and position of pins, and cannot be determined with any degree of accuracy. If the chord is continuous, the author's method will give a weakness of bottom flange at panel point; and if not continuous, a weakness of both flanges at center of panel. I think the only way we can know that the material is not overstrained is, first, to place the center of pin at the center of gravity of the section, as the effect of eccentricity in a continuous chord is difficult to determine; second, to consider as a beam of one panel length, subject to the maximum bending that will result from wheel loads and floor system, the beam to be considered as supported at the ends for center section and fixed at the ends for end section. I would use the working

stress a in both cases, unless the direct thrust is in excess of the bending stress.

The rule given by the author for excess of material in the heads of eye-bars does not agree with the results of experiments at the Keystone Bridge Works. We have found beyond question that the larger the pin as compared with the width of bar, the less the excess required to develop the strength of bar. The author makes the allowed bearing between pins and pin-holes a function of his compressive unit stress a. This is very troublesome to apply, and I believe a uniform unit stress of, say, 12 500 pounds per square inch, will give fully as good results. He makes the maximum allowed fiber stress on pins vary with the maximum tensile stress a. I think this could remain constant without detriment to the work, and be simpler in application. W. S. Thompson, Manager of the Dominion Bridge Works, as the result of eight or ten careful experiments on small scale machine for fiber stress necessary to produce permanent set in iron and steel pins, found that set was produced in iron pins at a stress of from 45 000 to 47 000 pounds per square inch, and in steel pins at a stress of from 70 000 to 75 000 pounds per square inch; the length was taken in clear between supports, and the applied pressure was on a bearing $\frac{3}{8}$-inch wide at center. I have seen pins taken from old bridges after many years' service, and apparently uninjured, which, according to the ordinary way of figuring here, sustained a stress of 100 000 pounds per square inch and upwards. The usual requirement of considering all forces applied at bearing centers, frequently leads to very erroneous results, and certainly should not be followed in all cases. I am satisfied, however, that good sized pins are essential in equalizing the stresses on connecting members, and that an allowed maximum unit fiber stress of 15 000 pounds for iron, and, say, 21 000 pounds for steel, is good practice.

The author requires double nuts on upset rods. I do not think this desirable; a single nut locked by a dap with a narrow chisel at the point where the thread enters the nut, being cheaper, neater, and perfectly effective.

The author requires that rivets shall not be spaced further apart in line of stress than twelve times the thickness of the thinnest external plate connected. If the space meant is from center to center of rivets, I think sixteen times the thickness of plate is sufficient, and will give better work.

The author's rules for the use of lattice bars are simple and practical, and a vast improvement on the rules sometimes given, which to apply require more labor than all other calculations combined, and which, in my opinion, are utterly worthless in theory, and result in great waste of material. Lattice bars should be heavy enough to protect the member from injury in handling, but I am satisfied small sizes are sufficient after the member is once in position. Several years ago I witnessed an experiment at the works of the Louisville Bridge and Iron Company on a post consisting of two channels connected by light lattice bars. I have not a record of the test conveniently at hand; but, quoting from memory, the post began to fail about axis through center of web at a stress somewhat in excess of formula requirements, and recovered on removal of pressure. After repeating this three or four times, with about the same result, it was decided to try the effect of lattice bars. At first one or two were knocked off, and the pressure was applied without change in result; and this was repeated, removing additional bars at random after each trial, until wide and irregular gaps were left between flange supports, and at last the post was destroyed by buckling of flange. My rule has been to provide lattice bars as follows: $2\frac{1}{4}$ by $\frac{3}{8}$ for $\frac{3}{8}$-inch rivets; 2 by $\frac{5}{16}$ for $\frac{3}{4}$-inch rivets; $1\frac{3}{4}$ by $\frac{1}{4}$ for $\frac{5}{8}$-inch rivets; and $1\frac{1}{2}$ by $\frac{1}{4}$ for $\frac{1}{2}$-inch rivets; which is believed to be good practice.

The rule given by the author for the length of the tie-plates at the ends of compression members—using the ordinary bearing and shearing values of rivets—results in extremely long plates. These not only add quite materially to the cost of structure, but are troublesome to paint. I believe a good rule is to make the length of connecting plates equal to the depth of member, and I have no evidence that longer plates are of any advantage.

The author makes no allowance for the web in the calculation of the flange sections of plate girders. I think one-sixth of web should be counted as flange area in each flange. It is undoubtedly effective to this extent, and not to consider it is discriminating in favor of lattice girders, which, in my opinion, are much inferior. In such girders the connection of elastic web members to comparatively inelastic chords necessarily gives very unequal stresses on the rivets, of which there is usually a deficiency at best.

The author remarks that the gain from the consideration of web is

very small; but as it amounts to about $2.50 per lineal foot for his double-track bridges, it seems to me quite an important matter.

The author specifies that the flanges of plate girders exceeding 12 inches in width shall have at least 4 lines of rivets. I do not think it advisable to use more than 2 lines of rivets under any circumstances. There is usually no difficulty, however, in limiting the width to 12 inches. I think the compressed flange of beams and girders should be computed by Rankine's formula, as given by the author, except I would use a constant numerator, the resulting working stress not to exceed the amount given by Launhardt's formula.

To simplify calculation, and to allow for the use of tables, which are of the greatest assistance in design, I think the shearing and bearing values of rivets in plate girders should be constant, and that an allowance of 7 500 pounds per square inch for shearing, and 15 000 pounds per square inch for diametrical bearing, will give good results. As the rivets in all probability are rarely ever subjected to any shearing, or bearing either—the friction between the plates due to the contraction of the rivets in cooling, being, by various authorities, more than sufficient to prevent it—I do not think any great fineness of calculation necessary. It is not usually considered good practice to rely to any great extent upon friction, but it nevertheless exerts a very large force in favor of stability; and at least one very high authority thinks we are justified in making use of it if necessary.

The distribution of rivets as specified does not differ materially from ordinary practice, but does not appear to have been observed closely in the standard plans of the Pennsylvania Railroad; for instance, on standard plan of 75-foot girders the flange angles are connected to the web by 2 rows of ¾-inch rivets staggered, the rows having a pitch of 4 inches and 8 inches respectively. The flange plates and angles are connected by 4 rows of ¾-inch rivets, 2 rows having a pitch of 4 inches, the other 2 a pitch of 8 inches. On standard plan of 44-foot girders, a 12-inch top plate has 4 rows of ¾-inch rivets pitched 2 inches transversely. The number of rivets provided in these girders appears to be much in excess of specification requirements. Rivet holes are a source of weakness, and an excessive use of rivets not only adds largely to the cost of work, but in my opinion is a material injury to the girder.

I do not agree with the author on his theory of the stresses in the webs of plate girders, nor see how it can possibly be true unless the

web is disconnected between the lines of his principal stresses. I am satisfied that the theory of Professor Rankine and others is nearly correct. It is quite clear that in a girder in which the great mass of the material is concentrated in the flanges, the intensity of the shearing force is practically constant throughout the depth of the web, and that all the forces acting on any material particle can be resolved into two forces, one of tension and the other of compression, acting at angles of 45 degrees with the neutral axis. It is not so clear at first glance that the intensity of this shearing force, horizontally, vertically and diagonally $= \frac{F}{h}$ per foot run, in which F equals the shear and h the effective depth in feet; but the consideration of a lattice girder makes this quite clear. Suppose a lattice girder loaded uniformly, and having struts and ties one foot apart horizontally, and inclined at angles of 45 degrees with the neutral axis. Let N equal the number of systems of bracing, then $\frac{F}{N}$ equals the average shear on each brace; but if the braces are of uniform thickness, like the web of a girder, the width of each brace is equal to $\frac{1}{\sqrt{2}}$ and the shear per lineal foot of width of brace equals $\frac{F \times \sqrt{2}}{N}$ but as the braces are inclined at angles of 45 degrees, the stress on brace equals shear $\times \sqrt{2}$ equals $\frac{2F}{N}$ per foot run of width, and as the braces are 1 foot apart horizontally and vertically $N = 2h$, therefore the tension per foot run equals compression per foot run equals $\frac{F}{h}$ which agrees with Rankine's theory. The distortion of the web under a load is such as would be produced by stresses of both tension and compression acting at angles of 45 degrees, and with compressive forces conveniently at hand it does not appear reasonable that the tensile forces will go out of their way in search of stiffeners upon which to act. Examples are on record which show that girders sometimes give way by failure of the web. The following are taken from Professor Unwin's Treatise on Iron Bridges and Roofs.

In the sixteenth experiment preliminary to the construction of the Conway and Menai Tubular Bridges, the weakness of the web was manifest, the sides buckling very much near the ends, the ridge being inclined to an angle of 45 degrees, as theory would dictate. In the thir-

tieth experiment a tube failed in a similar manner. In the fiftieth experiment on a 66-foot girder, 10 feet deep, the web began to buckle with 110 tons, and collapsed with 165 tons. In the second experiment on the model of the Britannia Bridge, the weakness of the sides to resist thrust led to the failure of the girder; the sides being thrown into diagonal waves of puckering; it would seem therefore that the compressive stresses in the webs of girders are not imaginary.

Although I believe Rankine's theory to be about correct, I do not think it necessary to make so much provision to meet the compressive stresses as he recommends. Stoney shows that a very small force applied at right angles to the line of stress is sufficient to keep a long strut in line. The moment the compressive forces deflect the web, the opposite tensile forces exert a powerful influence to bring it back, these forces being applied at every point of its length. In addition to this, the flanges resist to a considerable extent any tendency to buckle in diagonal lines, so that altogether it is not surprising that a very thin web will withstand the compressive shearing stresses safely.

In the designs of plate girders I have usually considered the length of web strut equal to the clear depth between flange angles, which I believe to give ample provision against buckling, and the stiffeners required by this rule are within the limits of ordinary practice. Stiffeners at a distance apart much exceeding the depth of girder, I consider useless, except to assist in straightening the web if kinked.

Girders formed with web and angles alone are prohibited. Such girders for short spans, such as track stringers, I consider desirable. They afford a better bearing for cross-ties than does a top plate with its lines of rivets, and greatly simplify framing. So far as I have ever heard, they do their work well. I think $\frac{5}{16}$ preferable to $\frac{3}{8}$ inch as a minimum limit for the thickness of webs in plate girders. The latter limit frequently necessitates considerable waste in material.

The author specifies that all truss bridges are to be cambered with a rise of not less than $\frac{1}{1200}$ of their length. I am in doubt whether he means permanent or figured camber; but, be this as it may, it appears to me that a rule based on the length of span only is defective. The amount of camber evidently should be equal to the deflection under a full load, plus set, that the chords may be straight and square bearing when subjected to a maximum stress. The general formula for the deflection of beams supported at the ends and uniformly loaded, when ap-

plied to open web-girders, may be reduced approximately to the form $d = \frac{l^2 s}{C h}$, in which

d = deflection in inches due to dead and live load.

l = length of span in feet.

h = depth of truss in feet.

s = mean stress per square inch in chords in tons of 2 000 pounds.

C = a constant.

Mohr's formula for deflection is as follows: $d = \frac{e\, u\, s\, l}{E}$, in which

u = stress in member due to 1 pound weight at center of bridge.

s = stress per square inch in member when bridge is fully loaded.

l = length of member.

E = modulus of elasticity of member.

This formula is supposed to give reliable results, and some actual recorded deflections certainly agree very closely with it. It is, however, troublesome to work, or requires more time than a busy bridge-engineer is often able to devote to it.

The formula first given is simple and easily worked. The value of C should be determined by comparing calculated and observed deflections, but for the want of a sufficient number of recorded examples it has been obtained by Mohr's formula in the following cases:

Name of Bridge.	Truss.	Mat'l.	l	h	d	$\frac{l}{h}$	$\frac{l}{d}$	s	C
Little River.....	Pratt.....	Iron....	99	24.5	0.882	4.0	1 347	4.20	1 904
Est.....	"	"	150	26.0	1.578	5.8	1 141	4.36	2 392
Tar River.....	Whipple .	"	196	32.5	1.801	6.0	1 306	4.43	2 906
Henderson	Warren...	"	246	31.0	2.427	7.9	1 205	4.15	3 333
Susquehanna ...	Whipple..	Steel ...	376	50 0	5.931	7.5	761	6.95	3 313
" ...	" ..	"	515	65.0	8.592	7.9	719	6.96	3 306
Henderson	Warren...	"	522	56.0	9.826	9.3	638	6.73	3 331

It will be observed that the value of C is nearly constant for spans between 246 and 522 feet, but falls off quite rapidly and uniformly for spans under 246 feet. Why it should do this I will not undertake to say, but assuming Mohr's formula to be correct, and allowing 10 per

cent. for set, practically the same results may be obtained by formula $d = \frac{l^2 s}{C h}$ by substituting for C the following values:

Spans up to 250 feet, $C = 900 + 8.4$ span.
" over 250 feet, $C = 3\,000$.

As this formula considers all the conditions, it is thought to be an improvement on the author's rule.

The author makes no greater provision for wind bracing than is made by many other engineers and much less than is made by some, nevertheless I have reason to believe that much material and money is being wasted in this direction. Up to about ten years ago it was not customary for bridge-builders in this country to figure wind bracing at all, but rods were put in of such size as was thought to be sufficient, ranging from 1-inch diameter, for spans of 150 feet and under, to 2-inches diameter for spans of 400 feet; or, say, about one-third average present requirements. These rods were frequently connected with the trusses in an insecure manner, usually at a considerable distance above or below the pin, developing large eccentric stresses, and often were ineffective, having nothing apparently to pull against, and these latter defects are also frequently found in much later designs. Thousands of these old and defective bridges are standing to-day; many have been taken down or strengthened on account of increased rolling loads; but very few have been blown down; and I know of no well authenticated instance of failure on account of insufficient strength of the lateral rods. Howe bridges have been blown down more frequently, but when we consider that they rarely ever have any portal bracing, or any provision for holding them up except the resistance at the joints of the lateral systems, which would hardly be considered in calculation; and the resistance of each truss to overturning about its own base, which does not cut a very large figure in calculation, it is not surprising that they occasionally topple over; this, however, is the exception, even when inclosed, exposing a full broadside to the action of the wind. Probably not one bridge in ten thousand is ever exposed to one-half of the force of wind that the specification considers. If the same rules were applied to buildings that are undertaken to be applied to bridges, but few could afford to build, and no contractor who would truly design and compete for such a building could hope to get a contract. The engineer who has had occasion to examine the exceedingly flimsy char-

acter of the great majority of iron buildings designed for mills and factories throughout the country, will readily admit that they are not in a condition to safely withstand any great amount of wind.

These insecure buildings, except in rare cases, appear to stand year after year without apparently any well-defined reason why they should, and it can hardly be claimed that human life is any more endangered by an insecure bridge than by an insecure mill in which hundreds of men are employed. These remarks are not offered as an apology for light and insecure work, for I would not be guilty of constructing it, and would freely condemn it wherever seen, but to call attention to what I believe to be a fact, that the danger to a bridge from wind pressure is much over-estimated.

Having intimated that the old bridges cited would fail under a wind pressure much less than that specified, it remains to prove that modern bridges would share the same fate notwithstanding the superfluous strength of the lateral systems. We will take as an average case the following example of a through bridge, Whipple truss:

Span, 201 feet 10¾ inches center to center of end pins, 11 panels.
Depth, 32 " 0 " " " chords, 20 feet clear.
Width, 17 " 9 " " " trusses, 16 "
Dead load, 1 400 pounds per lineal foot.
Rolling load = { 2–80.6-ton consolidation engines followed by 2 240 pounds per lineal foot.

Bridges unloaded, wind 50 pounds per square foot, as per specification.
Horizontal force at top of end brace 22 340 pounds.
Bending moment on end brace 270 314 pounds.

The fiber stress on end braces at bottom portal strut will be as follows:

From portal bracing..... 20 800 pounds per square inch.
" overturning tendency, 1 240 " " "
" dead load..... 1 980 " " "

Total.....24 020 " " "

24 020 + 6 110 = 3.93 times the allowed stress for vertical loading.

The following table shows the condition of the bottom chord of the windward truss.

Panel.	Tension.	Compression.				Force per Square Foot Required to Collapse.
	Dead Load.	From Bottom Laterals.	From Overturning Tendency.	Total.	Resulting Stress.	
First....	36 800	38 100	23 100	61 200	24 400	30.0
Second..	36 800	68 600	23 100	91 700	54 900	20.1
Third...	52 900	91 500	23 100	114 600	61 700	23.2
Fourth..	79 700	106 700	23 100	129 800	50 100	30.7
Fifth....	97 100	114 300	23 100	137 400	40 300	35.3
Center ..	106 500	114 300	23 100	137 400	30 900	38.7

It appears to me inconsistent to use for the wind bracing a unit stress of but 1½ times as large as for vertical loading, when stresses 3.93 as large are permitted in the end braces; also to consider a force of 50 pounds per square foot on the lateral systems when a force of 20 pounds per square foot will collapse the structure. If the track stringers are spread wide apart they will materially assist the chords in resisting reverse stresses, but when placed under or near the rails, as they frequently are, but little reliance can be placed on them. A consideration of the leeward truss, when the bridge is fully loaded, taking into account the overturning tendency due to bridge and train, would show that the bottom chord of that truss was much overstrained. Although the above showing is bad enough, it would be much worse for a deck bridge, as the distance between trusses would be less and the surface exposed to the wind at top much greater than in the case considered. It is true that the author provided for these contingencies in his specification, but they are not provided for in practice, nor can they be until the present style of bridge-building is materially changed.

It is not probable that a train will be running at a maximum speed when exposed to a maximum wind, or when the wind alone is sufficient to upset an ordinary passenger car; for this reason I do not consider it necessary to add to the strength of the lateral systems on account of centrifugal force.

In all cases where the rods have adjustment, five tons are allowed for initial tension. This adds quite materially to the cost of a bridge, and does not appear to me necessary. Wind stresses and initial stresses are opposed to each other. The bottom lateral-rods of a bridge are not adjusted until after the span is swung. The bottom chords are pulled

straight by virtue of the tension in them, and in adjusting the rods there is no good reason why those crossing each other in the same panel should not have about equal tension; and the initial tension on one rod is held in by the rod in the opposite direction. When the wind stress comes on it is evident that its first effect is to relieve the rod opposed to the transmission of shearing stress to the supports, and when the wind stress exceeds the initial stress the latter disappears. The top-chord sections are faced squarely, and their natural position is straight, and a less force is required to hold them in this position than in any other.

If it is found necessary to screw up the rods too tightly during the process of lining up, if not found relieved, they should be relieved before final adjustment; but as the top chord is less flexible than the bottom, it is quite probable that some rods will have more stress than the corresponding ones in the opposite direction; but in this case, only the difference in initial stress can be added to the wind stress. So far for all practical purposes I do not think it necessary to make any allowance for initial stress, except when in excess of wind stress.

The specification requires that eye-bars when tested to destruction shall always break through the body. I do not think any manufacturer has yet succeeded in making iron eye-bars that will invariably do this, and so long as the bars have the requisite strength, it appears to me a matter of little consequence where they break.

With the exceptions noted, I have no objections to offer to the author's specification, and it certainly contains many good provisions.

Geo. H. Pegram, M. Am. Soc. C. E.—A great deal may be said on Mr. Wilson's very interesting paper. No subject is more fruitful of discussion, and it is fair to expect that much difference of opinion will be developed, and to predict a long step in advance in this most important matter of bridge specifications.

In keeping with the thoroughness with which Mr. Wilson has treated the subject, I have confined myself to the question of live load, in which, I think, some improvement can be made.

The custom of using two engines, followed by a train of uniform weight per foot, has been quite universal for some time. Mr. Wilson now, very properly, proposes three types of engines, because either may produce the maximum strain, according to the span and panel lengths.

But the amount of work necessary to produce the strain sheet is greatly increased, and where a bidder is required to make a dozen bids to each contract that he secures, all accompanied by complete strain sheets, the question of calculation becomes a serious one.

The direct use of the engine-loading in any case involves a deal of work, is often inexact, and, it seems to me, rather unscientific.

Some years ago the usual loading was a uniform weight per foot of span for chord strains, and the same load headed by one or two engine excesses for web members. The uniform load was sometimes varied, according to the length of span, but the variation was rather arbitrary and could not well be made to include the effects of driving-wheel concentrations of engines; the resort to the engine-loading would seem very natural and justifiable, but it is in some respects very objectionable. The labor of calculation could not be made greater. If the results represented the maxima for other engines, or for the same engines in other positions (head to head for instance), there might be some satisfaction, but they do not. To illustrate: The centers of gravity of two engines, as commonly specified, are about 50 feet apart, consequently for 25-foot panels the maxima concentrations will occur at alternate panel points, and therefore on the same web system of a Whipple truss; but if the panels are 17 feet, the maximum concentrations will be three panels apart, and therefore on different systems, and will not secure the maxima strains. Another evil to which engine-loading subjects us, is the variety of engines used, differing so little as not to materially affect results, but still necessitating special calculations. I believe the assumed loading should be rather excessive. In that case there would be less reason for such variety. The difference in weights of bridges proportioned for the lightest and heaviest loading now specified is only about 10 per cent., and with the train weights constantly increasing, a little excess strength should not be objectionable.

The nearest approach to a uniform load will best approximate the various engines and panel lengths, and afford the greatest facility of calculation.

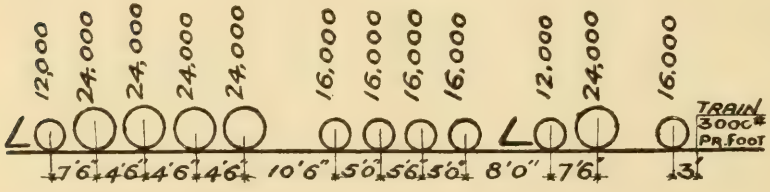
This loading should be modified to produce the effects of engine concentrations, and the actual engine loads should be the basis.

As fulfilling these conditions, I would submit the following specification as an equivalent of the loadings given in Mr. Wilson's paper, viz.:

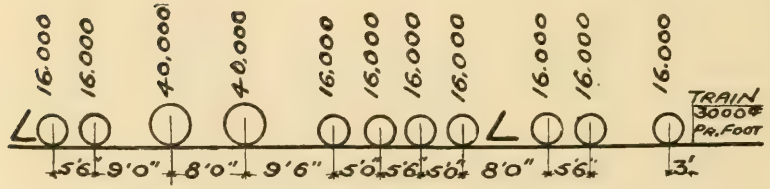
The live load on each track to be assumed in calculation, shall consist of a uniform load of 2 900 pounds per foot, together with a concentrated load of 25 000 pounds, the two to be combined in a way to give the maxima strains in all parts of the bridge.

A concentrated load passing over the length of the span is equivalent to a uniformly distributed load of double the amount; consequently, girders and the chords of trusses would be calculated for a uniform load of $2\,900 + \frac{50\,000}{S}$ (where S = the span) per foot of track. The web strains in trusses would be determined by adding 25 000 pounds to the common panel load at the end of the uniform load. This, of course, presumes that the uniform load shall extend over the panel in front of the concentrated load, and that the half-panel load at the advance panel point shall be neglected similar to present custom.

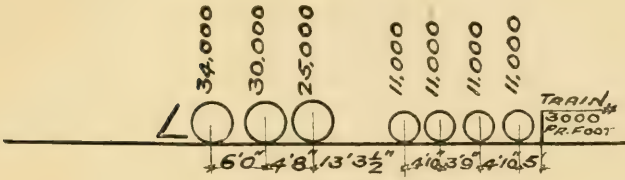
The following diagrams, showing the loadings taken from Mr. Wilson's paper, and the proposed loading, are introduced here for ready reference.



Typical Consolidation Engine Loading.



Typical Passenger Engine Loading.



Class "M" Engine Loading.

Proposed Loading (as equivalent of the above).

The following table gives the equivalent uniform loads for the several engines, obtained by moving the engines on the span to secure the maximum moment.

EQUIVALENT UNIFORM LOAD PER FOOT OF TRACK FOR GIRDER BRIDGES.

Span in Feet.	Engine.	For Middle Moment.	For Moment 10 Feet from End.	$2\,900 + \frac{50\,000}{S}$
8	Passenger	10 000	.....	9 150
8	~~Mogul~~ Class. M.	8 500	.....	9 150
8	Consolidation	6 000	.....	9 150
10	Passenger	8 000	.....	7 900
10	~~Mogul~~ Class. M.	6 480	.....	7 900
10	Consolidation	5 280	.....	7 900
12	Passenger	6 667	.....	7 070
12	~~Mogul~~ Class. M.	5 667	.....	7 070
12	Consolidation	6 000	.....	7 070
15	Passenger	5 333	.....	6 233
15	~~Mogul~~ Class. M.	5 933	.....	6 233
15	Consolidation	5 760	.....	6 233
18	~~Mogul~~ Class. M.	5 670	.....	5 677
18	Consolidation	5 333	.....	5 677
20	~~Mogul~~ Class. M.	5 460	.....	5 400
20	Consolidation	5 280	.....	5 400
30	~~Mogul~~ Class. M.	4 410	.....	4 567
30	Consolidation	4 480	4 560	4 567
40	"	4 070	4 134	4 150
50	"	3 712	3 940	3 900
60	"	3 547	3 760	3 733
70	"	3 438	3 633	3 615
80	"	3 370	3 543	3 545

The 8-foot span is so exceptionally short, that it is felt that the difference of 8½ per cent. may be disregarded.

In trusses, the panel length exercises such an influence on the strains that it was thought best to work out complete strain sheets. This has been done for a sufficient variety of conditions to show the general application of the loading proposed.

The class M loading was found so much below the others in the cases considered, that its results will not be given.

The Pratt (single intersection) and Whipple (double intersection) trusses were taken.

The strains are given for chords, end-posts and ties. For convenience of reference, the two end-chord panels (in which the strain is the same) are No. 1, the third panel 2, etc. The end-post is 0, the hip-suspender 1, the first inclined tie 2, etc.

The dead load is included in order to give total strains. It will be understood that the loading given is for one track, while the strains are for one truss, and therefore due to half the load.

CASE 1.—Pratt. 150-foot span in 9 panels of 16 feet 8 inches. Depth, 25 feet. Total dead load, 216,000 pounds.

	Chord Strains.			Web Strains.		
	Passenger.	Consolidat'n	Proposed.	Passenger.	Consolidat'n	Proposed.
0	.....	.....	.....	184 100	187 800	186 930
1	106 400	104 400	103 824	45 700	48 500	46 666
2	181 640	176 400	181 692	146 280	145 600	145 176
3	229 540	225 800	233 604	106 800	106 600	106 542
4	258 100	253 100	259 560	70 600	70 700	71 130
5	.....	.....	.....	37 700	37 900	38 940
6	.....	.....	.....	8 400	8 900	9 972

CASE 2.—Pratt. 150-foot span in 6 panels of 25 feet. Depth, 25 feet. Dead load, 216 000 pounds.

	Chord Strains.		Web Strains.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	.....	.....	204 600	204 510
1	146 000	146 062	64 200	66 700
2	228 000	233 699	133 000	134 026
3	257 400	262 911	70 600	72 100
4	.....	.....	16 600	18 607

CASE 3.—Whipple. 255-foot 6 inch span in 14 panels of 18 feet 3 inches. Depth, 29 feet. Total dead load, 591 000 pounds.

	Chord Strains.			Web Strains.		
	Passenger.	Consolidat'n	Proposed.	Passenger.	Consolidat'n	Proposed.
0	.....	.....	.....	382 080	381 100	378 506
1	204 000	203 500	202 141	56 900	59 100	60 062
2	297 700	297 600	295414 ~~295 414~~	178 500	177 000	182 127
3	446 800	442 000	450 869	204 100	202 400	208 768
4	575 800	572 500	575 233	170 800	169 400	175 132
5	667 200	662 100	668 506	134 300	133 300	138 450
6	724 500	719 400	730 688	104 200	103 500	107 855
7	754 500	751 100	761 779	71 000	70 700	74 216
8	.....	.....	.....	43 800	43 800	46 664
9	.....	.....	.....	13 200	14 000	16 067

CASE 4.—Whipple. 255-foot span in 10 panels of 25 feet 6 inches. Depth, 29 feet. Total dead load = 600 000 pounds.

	Chord Strains.		Web Strains.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	.....	.....	419 900	415 807
1	277 100	275 116	76 200	79 475
2	392 900	397 390	199 100	193 899
3	581 400	580 801	229 200	224 242
4	701 000	703 075	172 700	169 110
5	756 600	764 212	108 500	106 409
6	.....	.....	59 400	58 646
7	.....	.....	2 400	3 415

*CASE 5.—Whipple, 403-foot 9 inch-span, in 19 panels of 21 feet 3 inches. Depth, 42 feet. Total dead-load = 1 560 000 pounds.

	Chord Strains.		Web Strains.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	.....	.....	744 464	736 464
1	336 400	333 100	81 900	83 500
2	493 400	491 800	351 300	351 700
3	769 000	767 600	402 000	401 700
4	1 012 000	1 009 000	344 200	346 200
5	1 216 400	1 211 700	302 600	304 700
6	1 378 300	1 374 200	249 600	251 600
7	1 511 000	1 507 800	210 300	212 600
8	1 595 100	1 601 300	159 800	161 700
9	1 654 000	1 655 900	123 000	125 000
10	.....	.....	74 600	76 400
11	.....	.....	40 300	42 000

In Case 3 it will be observed that the tie strains are slightly less with the Consolidation than with the proposed loading, while in Case 4 they are greater by about the same amount. This is due to the manner in which the maxima engine concentrations strike different truss systems in one case and the same system in the other as previously explained.

* In calculating strains for uniform loads, it is not necessary to separate the systems except when the number of panels is odd.

The proposed loading is submitted simply as an equivalent to those given in the paper under discussion, and to illustrate the fact that a variety of engine and train loads may be represented by a uniform load in combination with a single concentrated load.

As a standard loading, I would suggest making the uniform load 3 000 pounds, and the concentrated load 30 000 pounds. This will cover an increase in the engine-loading which will probably take place at no distant day.

The Baldwin Locomotive Works build a Decapod locomotive, as it is called, which is heavier than any given in the paper. It has five pairs of driving wheels on a base of 17 feet, with 25 600 pounds on each pair, 16 000 pounds on the pair of truck-wheels, and 20,000 pounds on each of the four pairs of tender-wheels.

This engine is designed for mountain service, and may not come into general use. These engines would produce slightly greater strains than the suggested loading, but not enough, it is thought, to be taken into account until their use becomes very general. The difference would be greatest in some girder spans where the customary percentage added for impact would be excessive on account of their slow speed, and thus tend to equate the loads.

C. C. SCHNEIDER, M. Am. Soc. C. E.—I have read Mr. Wilson's paper with great interest, and consider it a valuable contribution to the profession.

Mr. Wilson has departed from the usual specifications, by using a modification of Weyrauch's formula for reducing the permissible unit stresses, and thus increasing the sections for impact. I have of later years used a similar formula for the same object. Not that I believe in the correctness of the formulas of Launhardt and Weyrauch as deducted from Wöhler's experiments, but as a convenient empirical method of increasing the sections for impact in proportion of the ratio of dead to live load.

There are a few points, however, in Mr. Wilson's specifications, on which I hold different views.

Like most other specifications, Mr. Wilson's gives three formulas; or rather, one formula with three different coefficients in the denominator for compression members, distinguishing fixed ends and hinged ends.

I have for several years past used only one formula for all compres-

sion members, regardless of their end connections, employing the one which is on the side of safety, viz., that which applies to struts with hinged ends.

A compression member in a structure is not in the same condition as in a testing machine; and while in a testing machine the struts with fixed ends show a greater strength than those with hinged ends, it is different when the struts are in a structure. The strut with the ends fixed in a testing machine is carefully adjusted, with the surfaces pressing against its ends exactly parallel and at right angles to the axis of the strut; while in practice, the strut with fixed ends receives its pressure mostly eccentric, and a bending strain at the same time. The conditions of the strut with hinged ends in the structure come nearer to those which prevail when the strut is in a testing machine than is the case with the strut whose ends are fixed. The strut with hinged ends will receive its direct pressure practically concentric in the structure as well as in the testing machine, and the bending strains, originating with the friction in the pins, must necessarily be considerably smaller than in struts with fixed ends.

I am therefore of the opinion that if any discrimination is made between struts of different end connections, it should be in favor of the hinged ends.

I perceive that this opinion is also held by other engineers. Mr. Theodore Cooper, in his latest specifications for highway bridges, uses only one co-efficient in the formula for compression members, regardless of their end connections.

According to my experience, this discrimination in favor of struts with fixed ends has done a great deal to encourage contractors to design details which must be considered bad practice, they taking advantage of the specifications for the purpose of saving material.

Mr. Wilson considers compression members which are continuous over points of support as hinged; this I consider a move in the right direction.

However, I fail to see the reason why Mr. Wilson follows the older specifications, and specifies a smaller unit strain for compression than for tension.

All experiments prove that the ultimate strength, as well as the elastic limit, are practically the same for compression as for tension; why then not allow the same working strain in both cases?

The usual explanation given by engineers who adhere to this practice is: The material in compression members buckles or cripples before rupture takes place; therefore, for compression members we take this buckling strength as a guide to determine the permissible unit stress, while for tension members we take the ultimate strength. A good rule, however, should work both ways. The tension members are also crippled before rupture takes place; and in this respect all members are alike. Compression, as well as tension members, are practically crippled as soon as their fiber strains exceed the elastic limit of the material. I think we should be guided by the elastic limit of the material in determining the permissible stresses more than by the ultimate or buckling strength; as in practice we do not intend to reach either, but we want to keep enough below the elastic limit to leave a fair margin for defects in the material and for ignorance.

I would also call attention to the following clause in Mr. Wilson's specification: "In all cases where the rods have adjustment, an addition to the above stresses of five tons must be made for initial tension." This appears to be inconsistent with the formula previously given; as the initial tension does not affect the maximum stress, but increases only the minimum stress on that member, and hence I can see no good reason for increasing the section on account of this initial strain.

Mr. Wilson allows a variation between the diameter of pin and pin-hole of one thirty-second of an inch; this, in my opinion, is more than should be allowed in good practice. I think the variation between diameter of pin and pin-hole should be limited to one-fiftieth of an inch.

Of the various existing specifications, the "Erie" specifications may be mentioned as among the most prominent ones. They were in existence previous to most of those enumerated by Mr. Wilson, have been used as the standard specifications by many railroads, and have been extensively copied by engineers.

T. C. Clarke, M. Am. Soc. C. E.—Mr. Wilson has aimed in this paper to present a specification which shall insure good results. He makes one grave mistake, which has been pointed out, but which it will do no harm to call to his attention again. He prescribes methods of manufacture, and then demands certain results. These two things are incompatible. Either can be insisted on, but not both.

He makes no mention of steel as a material of construction. This,

perhaps, adds to the dignity of his paper, but deprives it of much of its present value.

I am prepared to state that the following things can be demonstrated to be true:

First.—Steel can now be obtained of a more uniform and reliable quality for bridge construction than iron.

Second.—Bessemer steel can be obtained of as uniform and reliable quality as open-hearth steel, provided the ultimate strength does not exceed 70 000 pounds per square inch.

During the last fifteen years, the principal improvements in bridge construction are as follows:

I.—Use of better materials.

1. Substitution of rolled iron for cast-iron.
2. Substitution of steel for rolled iron.
3. More thorough system of inspection.

II.—Increasing the strength of different parts of a bridge, in direct ratio to the proportions of live to dead load, and to the direct action of live load upon them.

III.—Providing for wind pressure.

IV.—Better connections.

1. Better proportions of eye-bar heads and pins.
2. Use of pin joints at top and bottom of end posts.
3. Better attachment of floor system to trusses.

V.—Improvements in design.

1. Less depth of truss.
2. The general use of long panels.

Many other things will doubtless occur to readers of this paper.

Jos. M. Wilson, M. Am. Soc. C. E.—Mr. Cooper is somewhat in error in assuming that the paper in question was presented for the sole purpose of obtaining the benefit of criticism from bridge experts. While criticisms and discussions are very desirable, and are welcomed by me as tending to the ultimate of a standard specification for the American bridge engineer, yet my particular object was to offer the paper as a contribution to the profession, supposing that my long practical and successful experience, which is hardly as limited as one might be led to infer from Mr. Cooper's remarks, would give a reliability to my methods valuable to others who might be traveling in the same

direction. The specifications submitted are free for the use of any who may desire to adopt them, either as they stand or with whatever modification or improvement they may deem advisable, and I trust that they may be of some service to my fellow-laborers.

I adopted the formulas as advocated by Launhardt and Weyrauch, not because I believed them perfect or fully covering the case, but because, in my opinion, they were an approach to a more rational and proper method of considering the subject than anything that had as yet been brought to my notice. I believe that the true method in forming a specification of this kind is to generalize it as far as possible. I am glad to notice that the interesting nature of Wöhler's experiments is admitted by Mr. Cooper, and their correspondence with the fatigue to which material is subjected in actual practice. It is true they are somewhat limited in extent, and it is to be regretted that there are not more of them; but, so far as they go, they lead us to what we want, and I fail to see where Mr. Cooper proposes anything better or more reliable. If his great practical experience or numerous experiments have produced anything better, I have not as yet had it brought to my notice. The results of the formulas as given by me certainly compare very favorably with the practice of high authorities, varying generally on the side of safety, and they are undoubtedly more rational than the haphazard method so generally adopted. Plate No. XXXVII shows that the results are in harmony with the practice of different engineers by other methods, hardly any two of which agree. And as to formulas, each engineer has his own, and very little basis on which to found it. Take the formulas, for instance, proposed for alternate stresses by Mr. Bouscaren in his criticism. He may be satisfied with it, and he admits that he adopts it from Launhardt, but what have we in the way of experiment to confirm it, except by comparing it with what others adopt, and what authority have they for their assumptions. If they have made extensive experiments, we have not heard of them. All are empirical, and my effort has been to use a general expression that I thought would represent or come sufficiently near to the average of the best practice, and at the same time be surely within the limits of safety as known to us. I am confident that any comparison between the results by my method and any of the old methods, as advocated by several of my critics, will show to the advantage of my formulas. The remarks of Mr. Mace Moulton tend to confirm this. It is true that I have raised the working stresses

for compression; but this, I believe, is more in unison with the latest experiments, and many favor raising them still higher.

I admit the desirability of having regard to the elastic conditions of the material rather than to the ultimate strength, and I believe that my specification shows it; I cannot allow Mr. Cooper's objection concerning the use of these formulas, that anything is left in indefinite shape, or that varying interpretations can be made by different designers. The method, on the contrary, is rigid and exact, and if properly followed, must result in compelling a just and fair competition, and first-class work. If any one bidding under a specification fails to conform with it in every particular, it is the duty of the engineer receiving the bids to point it out and to insist upon a compliance; a matter of no difficulty if he understands his business and conscientiously follows it. I speak from considerable experience on this subject. Bidders can be compelled under this specification to proportion the work exactly as it is wanted, and there are no two or three interpretations to be followed. It will never be possible to prepare a specification on which an engineer, ignorant of the laws of bridge construction, can obtain true bids without expert advice. The plans should always be submitted to an acknowledged expert before adoption, and the work should be carried out under his direction.

I presume, as a fundamental starting point, that when bids on a bridge are received, the plans, details, etc., are examined by a competent and reliable engineer, who not only has authority to compel compliance with the specifications, but can also require proper and approved forms of detail.

My experience may, perhaps, have been somewhat limited. I can only say that it has been connected with actual bridge work for over twenty years, during which time more than 20 miles of bridges have been constructed from my plans and specifications. Most of my bridges, it is true, have been built from my own designs, but not all of them, and I have had to deal with contractors who were quite as ready to take advantage of opportunities as any. In these dealings I was entirely satisfied with the completeness of the specifications and their ability to cover the case.

In regard to impact, I am aware that we do not have very much data to draw from, but as far as the law has been known, I think it has been met. Prof. Robinson may throw some light on the subject, but if Mr.

Cooper, as he says, does "not know with sufficient definiteness the effects of impact upon a structure and its individual members to formulate any law," how can he adopt a rule-of-thumb method which will meet the case. We know at least that impact is a function of the live load; why not then treat it as such and let its effects vary accordingly? This is surely better than guessing at it for each individual member of the truss. The form and construction of any member certainly has a large influence upon its ability to resist stress of any kind, impact or otherwise, and it is best, that after we decide what stresses come upon these members, we should consider this question of form and also the mode of construction of the member, the number and disposition of its parts, rivets, etc., all of which is taken care of in its proper place.

The reason why girders formed with web plates and angles only, having no upper flange-plate proper, are not allowed, is largely a practical one. The upper web plate is a great protection against the weather, or the admission of water, charged, perhaps, with sulphurous or sulphuric acid, to the joints of the iron. In addition to this the plate stiffens the angles on opposite sides of the web very materially, making them act more thoroughly together.

In my criticism on Mr. Cooper's article "on the rules for pitch of rivets and thickness of web" in riveted plate girders, I merely give the example as an instance of how a strict rendering of Mr. Cooper's rule would operate. I did not mean to infer that Mr. Cooper would really place the stiffeners in that way, as I felt sure he would know better than to follow strictly his own rule.

In reply to Mr. W. Howard White's query as to the prohibition of allowance of web in calculating plate-girder flanges, I would say that I do include the web in solid rolled girders where the thickness is comparatively great, the whole substance of the cross-section in one homogeneous mass, and the amount of material generally in excess for shear; but in the case of built girders with the plate web riveted to the flanges, where the flanges are formed to take the horizontal stress, and the web the shear strains, I think it is better to keep each to its own particular duty without depending on the other, and it is practically on the side of safety to so consider it. According to the theory of some, this web already has to resist a tension in one direction and a compresion at right angles to it. To introduce a third force in a diagonal direction would seem almost too much, the argument, if extended, leading to a "*reductio*

ad absurdum" that a web plate can resist any number of forces of either tension or compression in any direction at the same time, provided they do not follow precisely the same lines, each force being up to the full capacity of the sectional resistance of the plate at right angles to it. It is only of late years that I have required in plate-girder work, flange plates in compression, where spliced, to be covered to their full section, as it has been found almost impossible to obtain truly butted connections. Broken joints in built compression members, as posts, etc., where planed and well butted, would only need plates sufficient for proper stiffness.

I see no reason for increasing the number of ties under the rails beyond what is necessary to furnish proper support for them, and 20 inches to centers is a convenient distance for spiking. In case of wheels getting off the track they will run over these 10-inch clear spaces without difficulty, as practical experience has proved it. The outside timber guard-rail, well notched to the ties, is considered necessary to prevent the ties from crowding together, as well as a guard. The inside guard-rail may be necessary in certain special cases, but generally it is an open question whether it does not increase the danger from catching brake blocks, etc., between it and the regular rail and running a train off.

As to the limit of pressure on masonry, I think 300 pounds per square inch (21.6 tons per square foot) is sufficient for ordinary work. I have provided for a larger amount being allowed in specially authorized cases where it is expedient. I think 21½ tons per square foot is about twice what it should be for fairly good brick-work. Thurston permits on brick-work in cement a load of 10 tons per square foot. Trautwine states that ordinary brick-work cracks with 20 to 30 tons; good brick-work in cement 30 to 40 tons; and that first-rate brickwork in cement will crush with 50 to 70 tons per square foot. One-sixth to one-tenth of these figures would be considerably below what Mr. White gives. Sandstones and ordinary good local building stones are very variable in crushing strength, and I do not think that the limit given in the specification is too small for general cases, particularly when the mason-work is not under the control of the bridge-engineer, as is often the case.

In regard to the use of paints, I did use red lead for some years as a priming coat at the works, and while I believe it to be of excellent quality for this purpose if properly put on, yet there was considerable difficulty in getting it applied. Humber, in one of his works on bridge

construction, very strongly recommends a special preparation of iron, and condemns all lead paints. Further information on the paint question is very much desired. An investigation into this matter in a chemical laboratory on a prominent railroad developed the fact, I believe, that paint made from the refuse of zinc batteries of the telegraph service was the proper thing to use, but every one knows that zinc paints will not stand the weather.

In reply to Mr. Burr's objection to my neglect of the load in advance of the drivers, I would say that I do not neglect the load ahead of the drivers; but in calculating web members I intend that the drivers shall be so placed as to give a maximum cross-girder load, and any load on the previous cross-girder is omitted, as I believe this will give maximum effects in the web members.

Concerning the matter of web stiffeners I of course admit, as I stated in my paper, that a web of sufficient thickness compared to its depth will resist by compression and tension just as Rankine computes it, but when the question of stiffeners comes to be considered, it is for webs very thin compared to their depth. In the model which I used, the web was of thin letter-paper with practically no stiffness in it at all. The stiffness which would be imparted to it by tension in the other direction is of no importance, as a deflection ruinous to the bridge would be reached before it could come into effect. Take a long, thin bar under tension. How much force is required at right angles to its length, applied at its center, to give it quite a sensible deflection? Its own weight will do it. If we assume that the stiffeners are not fastened to the web and have no end bearing, but are merely held against it so as to prevent any motion in a direction perpendicular to its plane, we would then have the action as described by Prof. Rankine, in which the web would be considered as a fixed column in a diagonal direction between stiffeners, of a length equal to the diagonal, and the action as described by Mr. Burr would then take place. The computations, followed strictly according to Rankine, would deduce a thickness of web that would be out of all proportion to practice. To keep that thickness down to reasonable limits, Mr. Burr would have to place his stiffeners very much closer than practice requires. If, however, the stiffeners are riveted to the web and have end-bearing under the flanges, compression to the web cannot take place without a resistance from the stiffeners, the strain being transferred to them by the tension on the web, either

through the rivets or *via* the upper flange. This action may be more or less partial depending upon the thickness of the web, but to avoid uncertainty I consider it best to put sufficient section in the stiffeners to take it.

Consider a lattice girder with a number of intersections, the web members running in opposite directions, inclined at, say, 45 degrees with the vertical. Those taking tension are computed as such, those resisting compression are computed as struts. Now let the compression members be placed vertically and the tension members be left as they are. The same condition applies, remembering that some diagonal members must exist in both directions for tension from direct and counter stresses. Now develop the lattice into a plate-girder by making the tensive members of the web continuous, and by concentrating the compressive members into vertical struts at regular intervals apart, equal to, say, about the depth of the girder, we have the condition of a plate girder calculated as per my method. If the original lattice with its web members remaining at 45 degrees were developed into a plate-girder in the same way, the compression struts would be at 45 degrees instead of vertical, and if both tensive and compressive members were developed into plates, the condition would be that of a plate girder without struts, and would be calculated as per Rankine's method. The natural direction for the forces to go would be in the inclined direction, and, theoretically, it would be best to put the struts inclined, but they are placed vertically as a practical matter of facility in manufacture and in making of connections. A portion of the web held between the struts, and extending for a short distance outside of them, depending upon its thickness and the question of its liability to buckling, may act with the strut to take compression. I do not wish to be understood as denying that a plate will take compression and tension at right angles on the same section, but with deep webs it is safer not to go down too close in these matters.

I beg to thank Prof. Merriman for his beautiful exposition of a method of determining the working stresses for members subjected to varying stresses. I have not seen his method of deducing his formula, but it certainly appears to give very rational results.

I have read Prof. Robinson's criticisms and recommendations with great interest. His paper requires considerable study, and I am at present, for want of time, hardly prepared to say how it might influence

me in the matter of modifying my specification in the future. His first recommendation is one, however, not very difficult to provide for, and well worthy of consideration. There is some risk in lessening the allowance for safety until we do know all the causes which we have to provide against. We must be sure that we eliminate all the ignorance before we ignore the co-efficient of ignorance. It would be very satisfactory to know positively all the stresses to which a bridge may be exposed, and we would like to take advantage of any rules in arrangement or distribution of parts that would tend to lessen, for instance, the effects of impact. But practical experience with what we call our good bridges would still warn us that the results should not give us lighter ones, unless it can be proved that the arrangement of panels, or other means of avoiding certain actions of forces, is a positive improvement for general cases. A law for the lengths of panels might be adopted to meet certain standard wheel spaces, and bridges erected in conformity with it. Then, some time hereafter, it is more than likely that new designs of cars to suit particular requirements of trade would be put into service without any consultation or even notification to the engineering department, the proportions of which would eliminate all the advantages derived. Bad joints in the track would be a source of increase to the percentage for impact that could hardly be estimated. Experiments recently made in Europe by M. Considère in reference to increased stresses in members of bridges from passing trains, showed a marked increase in the stresses from bad joints in the rails. It seems to me also that the speed of trains would affect the amount of impact very materially, and that this would be a difficult force to take account of with any exactness, as, in the case of freight trains particularly, the speed is very irregular.

The effect of the unbalanced engine wheel, as deduced by Prof. Robinson from his experiments, showing, as it does, a material increase with the length of span, is so at variance with the generally accepted practice, that it will bear considerable examination and investigation before being adopted.

Very often when bridges break down, facilities for getting at the real cause, or of forming an opinion on the same, are not afforded by the railroad company, in order to avoid the responsibility that might ensue were the true cause known. I believe that in all cases of failure, if the full details of the structure and the loading can be obtained, it will be found that some part of the bridge was not designed in consonance with

the recognized practice of the present day among first-class engineers. The mystery with which the case is often shrouded is only a cover for the protection of the railroad company.

If we know the true amounts of all the forces to which a bridge can be subjected, and raise the unit strain accordingly, lessening the factor of safety, the result would, I think, not give us essentially lighter bridges as a class, although better designed and with a more uniform proportioning of parts, as we would have greater stresses to take account of than we know now. Of course the factor of ignorance would be lessened to a minimum, and this would be much more satisfactory. Our experience shows us that there is a large factor needed for wear and tear, and also a margin for the future exigencies of the service. We have never yet been able to anticipate the future sufficiently to allow our railroad bridges to last to anything like the limit of the durability of the iron. The material may last indefinitely, but in Europe, as well as in this country, bridges which have not worn out are constantly being replaced, solely because they are too light for the increased service.

Professor Robinson takes some exception to the values in the formulas of Launhardt, Weyrauch, etc., which would open up a discussion, going behind my assumption, and raising a question as to the correctness of the deductions of eminent authorities, which I will not now attempt to consider. Such a discussion would be very interesting if it could be followed out, and might result in modifications of value.

In reply to Professor Swain, I would say that, so far as I am prepared to say now, I would not like to increase the stress for iron under quiescent loads to a greater amount than 15 000 pounds per square inch. Professor Cain's method of allowing for impact appears to me to give results in consonance with practice, and it is very convenient of application. Professor Robinson's researches are throwing a good deal of light on the impact question, and I am hardly prepared to say just now what ultimate view I may take of the matter. In regard to the formula of Winkler quoted by Professor Swain, a comparison with my formulas using the same limiting stresses for all live and all dead-load as I do, would give results for intermediate cases not varying very essentially from mine. But with the values Professor Swain suggests for American practice, the results would be considerably above what I would like to adopt.

Professor Swain's suggestion for calculating the upper chord when

subjected to cross-strain is not tenable, as the chord does not act as a supported beam, and it must be taken in the condition in which it actually exists. The moments may be positive or negative according to the loading. Some approximation may be used in obtaining the required values, but we should endeavor to obtain the maximum effects.

Mr. Moulton's deductions and comparisons are interesting as showing the correspondence between the formulas of the specification and practical experience. The question of raising the value of the working stress in compression to that used in tension is a matter that will bear consideration, but I do not think we have sufficient data as yet to fully justify this departure, although I can signify my inclination in that direction.

Replying to Mr. Boller's criticism concerning continuous compression members being hinged over points of support, I think the same principles would apply as in continuous columns stiffened at intermediate points. A pin, while it would hold the column in place, would give no bending resistance around an axis through its center. If the chord is so loaded that adjoining bays tend to bend in the same direction, then the assumption does not apply, and in the case of an upper chord of a through bridge, the moment due to the weight of the member itself would probably have this effect. But for deck bridges with a heavy load on one bay and only the dead weight of the member itself in the next, the downward moment of the latter is easily overcome by the reflex moment for the adjacent panel.

Concerning the criticism of Mr. William Sellers in reference to the use of double-rolled iron and the exclusion of scrap-iron, I would say that, as a general rule, I am myself opposed to specifying methods of manufacture at the same time that certain conformity to physical tests is required, and some twenty years ago modified the specification then used on the Pennsylvania Railroad very materially in this respect; but the requirements of double-rolling and omission of scrap was retained at that time, because I believed that in the then state of the manufacture we obtained a better quality and more uniform material than we could otherwise do. These restrictions have been retained ever since, although I am willing to acknowledge the arguments on the other side, that some iron is better not double-rolled; that the manufacture has progressed so that there is no difficulty in meeting the specification without this requirement; that scrap has to be added to some iron to obtain

the wished-for result; that the manufacturer in his own interest will not use short or improper scrap, the increased facilities for testing, etc., making it too much risk to do so; and also, above all, that it is extremely difficult, if not impossible, to know that we obtain what we specify, it being almost certain to be otherwise, to the detriment of conscientious bidders for material, etc. Bearing in mind also the greater advantages and reliability of the methods of testing employed of late years, I am perfectly willing to modify my views in this respect, particularly so as it will tend to promote harmony of action.

The proportions for test pieces adopted were believed to conform to those used by the United States Government Inspectors, and it was considered exceedingly desirable that there should be a standard in this respect. I admit the advisability of having the length a uniform multiple of the shortest transverse section, and would willingly approve of a common standard for test pieces if one can be agreed upon.

As to the limitation of the specification to wrought-iron alone, I would say that I have not the data at hand to take up the question of steel in the way I should like to treat it. My experience with steel has not been very great, and while I should not hesitate to use it for heavy compression pieces or large chord links, especially in a long span bridge, yet I hardly think I know enough about it to attempt to form a general specification for it.

Mr. Thatcher's suggestions in reference to the simplification of the work of computation are certainly worthy of consideration. I am uncertain as to whether a uniform load can always be found that would produce essentially the same results as the wheel loads, but I have no doubt that tables could be prepared covering the maximum effects for the several classes of engines, which would facilitate work. We have already tables in use which cover a great many of the calculations of the specifications.

Concerning the rule that the cross-girder load shall be considered as the head of the train in proportioning web-members, and Mr. Thacher's criticism that the percentage of gain on the different members varies largely, I would say that I do not think this variation amounts to more than the variation in the actual weights of different engines of the same type from the assumed weight for that type. There will always be a slight gain in stress in the member which is a maximum, but I think this will not exceed the variation for different engines.

As to the division of dead-load at panel points, I agree that an approximate general rule might be an improvement.

I quite agree with Mr. Thacher that it is poor construction to carry the floor of bridges on the upper chord between panel points, but it seems difficult to avoid it in many cases. The determination of the stresses from the two kinds of loading is undoubtedly involved in some uncertainty, but I have endeavored to cover the case in the method I have adopted. I have treated continuous chords only, and my intention is that the combined stress in the outer fibers shall not exceed the column stress b at the middle of the panel, or the compression stress a at the panel point. I found no reliable formula for giving the effects in a continuous chord due to the moments from pins being out of the neutral axis, and therefore had some investigations made which proved that while the moment from each pin affected every panel towards the middle of the span, the alternate panels having moments of different signs, yet the combined results amounted to very little except in the first and second panels and the pin point between them. I prefer, however, to avoid these moments as far as possible by keeping the pins very near the neutral axis, and for any slight effect in the first panel there is generally an excess of section over the theoretically required amount. I have considered the dead load as acting on a continuous beam, and the moment for live load as three-fourths that on a supported beam, believing it to be a close approximation to the effect from a single rolling load on a continuous beam. The section thus given for the middle of the panel is generally in excess of that required at the panel point, so that it was not necessary to specify that the value of a should not be exceeded in the lower flange at that point.

The question of excess of material in the heads of eye-bars is undergoing considerable modification from the old rules, as I intimated in the body of my paper. I am not prepared to say now what modification may be made, and probably no general rule would apply to all methods of manufacture. The essential point however is, that the head should be fully as strong as the body of the bar, and the less metal that can be used for this purpose the better.

The use of constant values for bearing stress and for bending stress of pins might give less labor in making calculations, but I do not think this of sufficient importance to allow us to ignore the differences which would then occur with extreme values of the unit stress.

The limiting distance of twelve times the thickness of the plate between centers of rivets is based on the rule that the compressive strain in the flange is limited to that for a length of twelve times its width.

I do not believe in having the tie plates at the ends of compression members too long, but I think it well to keep up the number of rivets as the specifiation requires. Future experiments may cause some modification of my views in this respect.

In the special cases of standard girders mentioned by Mr. Thacher, I would say that where rivets are staggered in rows near together, I would not require those of each row to be kept twelve times the thickness of the plate apart. I think that compression flanges should always be well riveted together, and I object to separate plates or pieces unless they are so riveted as to allow no tendency to separate action. The 44-foot standard girder mentioned, was designed some years ago, before the present specifications were prepared, and does not conform strictly to them in their present shape, as some modifications have been introduced.

The matter of camber is to my mind a question of appearance only, and generally I have the camber framed out in the track so as to make it level. I do not like to see a bridge fall below a level line and the specication gives a minimum rise only. The rule of the specification is, I think, sufficient for practical purposes.

I do not make any claim to originality in the formula for wind bracing, having adopted that which seemed best to suit my views. My experience with old bridges, however, has shown many cases in which the bracing was too light for ordinary use without any extraordinary wind action, in many cases it being impossible to keep the requisite lateral stiffness in the structure, the rods stretching continually under ordinary traffic from the action of other forces than wind. The assumed wind force is taken as a maximum, and of course it is not as severe in some localities as others, but it would be hardly possible to make any distinction for location, unless it were for a place where very extraordinary precautions would have to be taken.

Mr. Pegram's discussion on live load really proposes a more expeditious way of arriving at the same results as the specification requires. In the old days of my labors in the profession, I used, in common with other engineers, a load per foot lineal of each track to cover all cases. This method was improved upon by taking account of the engine wheel-loads. It was found, however, that no sooner was the latest and

heaviest engine in use on the road adopted as a standard, than a still heavier engine was brought forward. To provide for this, typical engines were assumed, which were supposed to cover what would be the actual increase in weights of motive service for some years to come. These typical engines had been in use for only a few years, when the third engine of the specification was brought into service, and it was found that some of its effects on cross-girders and for short spans were in excess of those from the typical engines. I therefore added it to the list. Its effect is limited, and only comes into play in a few cases. I have no doubt that half a dozen types of engine could now be found which exceed our typical types, although, perhaps, not much. This only shows how inexpedient it would be, even with more scientific and exact methods of calculation, to lighten our bridges. The great point in favor of stone bridges, and really their saving clause, has been their great excess of strength. Those who favor stone bridges say: Oh yes, they cost more money than iron bridges, but they are so much more permanent. I say, put the same excess of material in your iron bridges, or rather the same excess of cost, and you will have a structure much better adapted to resist travel, time, and the elements, than you have in a stone bridge. I have never been in favor of stone bridges of any size for railroad traffic, particularly in our climate. The material, from its brittle, crumbling nature, is very ill-adapted to resist the action of variable loads, impact, etc., and wherever frost can enter, it slowly but surely does its work. I have examined arch culverts over many miles of road; I have watched carefully some stone spans of considerable size, and have had to do with extensive repairs. I know that this is so, and the dilapidated condition of many even large culverts on some of the roads, is a disgrace to them. It is fortunate that they are below and out of sight.

To come back to Mr. Pegram's discussion, however, there may be some difficulty in arriving at an entirely satisfactory solution of the maximum effects by a simple system, but I believe it to be well worthy of the attempt. In using one set of engines similar to those in my specification, it is easy to make out equivalent uniform loads, giving the same moments as the wheel loads, and also the panel loads for all lengths of panels. Therefore, I did not appreciate the necessity of simplification so much as manufacturers would do, who have to deal with a variety of specifications. Any system covering the maximum

effects would have to be somewhat in excess in some members. I am hardly prepared to say how far Mr. Pegram's loading will satisfy the case. This will require some investigation.

In reply to Mr. Schneider, I would say that it is very seldom columns in bridges are other than hinged, but the specifications were intended to be general and to include all cases. Sometimes fixed ends do occur; but when doubtful, I should certainly consider the columns as hinged. I do not quite agree with Mr. Schneider on the question of initial tension as not affecting the maximum stresses in counter rods. We know from practical experience that when counters are screwed too tight in a bridge, the loading upon them will break them, and evidence of this often occurs. The initial strain screwed into the rod while the bridge is at rest unloaded, it seems to me will remain there, in addition to the counter strain that is put upon it.

When Mr. Clarke states that I make a grave mistake, I presume he refers to the question of double-rolled iron and the omission of scrap, a matter to which I have already replied. I am hardly prepared, however, to admit the great gravity of this mistake, although the requirement may be inexpedient.

In conclusion, I beg to thank those who have taken part in the discussion for the attention they have kindly given to the matter, and for the many valuable suggestions they have made. I trust that the result will ultimately tend towards truer methods of bridge calculation, and to greater uniformity in specifications.

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